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FIELD COMPARISONS OF HYDRAULIC CONDUCTIVITY METHODS AND DRAINAGE ENVELOPE HYDRAULIC CHARACTERISTICS

BY

DAVID E. KRAMER

A thesis submitted
in partial fulfillment of the requirements for the
degree Master of Science, Major in
Agricultural Engineering, South Dakota
State University

1974

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FIELD COMPARISONS OF HYDRAULIC CONDUCTIVITY METHODS

AND DRAINAGE ENVELOPE HYDRAULIC CHARACTERISTICS

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Thesis Adviser Date

Head of Major Department Date

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INTRODUCTION

Drainage of irrigated lands may seem to be a paradox when the basic problem associated with irrigated lands is a shortage of soil moisture. However the water regime under irrigated conditions is often drastically different from corresponding dryland farming conditions. The soil moisture content is greater under irrigated conditions than dryland conditions, which establishes a field condition where downward movement of water below the root zone can occur. The downward movement of water called deep percolation may be planned to leach salts out of the root zone or may be caused by improper irrigation management. Since many irrigated lands do not have adequate natural subsurface drainage to control this deep percolation, a drainage system is often required.

South Dakota is presently planning for the irrigation of approximately one-half million acres of land in the proposed Oahe Unit located in the north-central part of the state. The main body of land in the Oahe Unit is the Lake Plain area which was once the lake bed of post-glacial Lake Dakota. Approximately one-fifth of the total cost of the project will be for drainage systems (United States Department of the Interior, 1965).

The Bureau of Reclamation is responsible for the design of the Oahe project. They are presently using a transient drainage spacing equation developed by Glover (Dumm, 1954) in the design of the drainage system. The use of this design equation requires the evaluation of two soil parameters; saturated soil hydraulic conductivity and drainable porosity. The auger hole method is usually used to determine the saturated soil hydraulic conductivity under saturated soil conditions. However, much of the Oahe Unit does not currently contain saturated soils in the region of the soil profile that will control the flow of water to the drains. The Bureau uses a point field method called the pump-in method to estimate the saturated hydraulic conductivity of the soil under unsaturated conditions. The accuracy of the pump-in method as compared to the auger hole method has been questioned. Therefore, the Bureau of Reclamation and the South Dakota Agricultural Experiment Station decided to compare these two methods on an experimental plot at the James Valley Research and Extension Center which is within the bounds of the Oahe project.

This experimental plot area contains a subsurface drainage system which was installed in 1967. Since the soils of the plot area are not completely uniform, the drain line discharges reflect an average of the soils and envelope hydraulic characteristics. An open trench construction technique was used to install the drain lines which resulted in a disturbed soil profile above the drain lines. Drainage design equations in contrast assume a homogeneous soil which means the disturbed trench soil has the same drainage characteristics as the undisturbed soil. There is a possibility that the trench material can be more permeable than the undisturbed soil and act like an open trench or it can be less permeable and restrict the flow to the drain line. Therefore an investigation of the disturbed soil hydraulic conductivity was initiated to determine the effect of the trench backfill material on drain line discharges.

Envelopes around the drain lines can also affect the ability of water to enter the drain lines. The envelope materials are designed to be more permeable than the surrounding soil; however, this may not always be the case. The drain lines in this experimental plot area are surrounded by two experimental envelopes (Figure 1); a three inch all gravel envelope and a combination three inch gravel and fiberglass envelope. The relative hydraulic characteristics of these envelopes under field conditions were investigated to evaluate the functional adequacy of the envelopes.

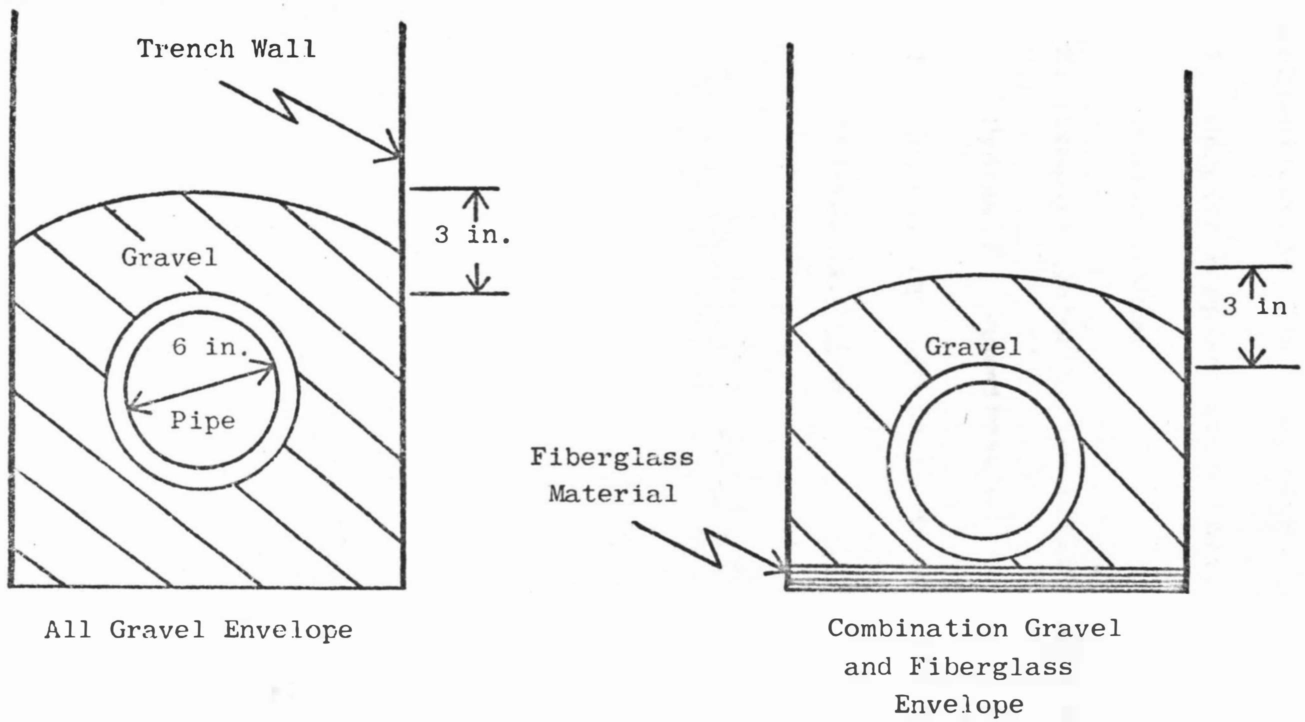

Figure 1. Drainage Envelopes Installed in the Field Experimental Area

OBJECTIVES

The objectives of this study were:

1. Compare pump-in, auger hole, and drain line hydraulic conductivities.
2. Compare disturbed (backfill) and undisturbed soil hydraulic conductivities.
3. Compare the hydraulic characteristics of two experimental envelopes.

LITERATURE REVIEW

This literature review is divided into two parts: a discussion of saturated soil hydraulic conductivity and drainage envelope studies.

Hydraulic Conductivity

The hydraulic conductivity of a soil is defined by Hillel (1971) as the ratio of the flux to the hydraulic gradient. Flux is defined as the volume of water passing through a unit cross-sectional area (perpendicular to the flow direction) per unit time.

Darcy's law. Henry Darcy (1856) developed an empirical relationship which is regarded as the fundamental law concerning water flow through soil. This law states that the flow of water through a porous medium is proportional to the hydraulic gradient and the hydraulic conductivity. This is symbolized by:

$$Q = KiA \tag{1}$$

where Q is the volume flow rate (L^3/T), K is the hydraulic conductivity (L/T), i is the hydraulic gradient (L/L), and A is the cross-sectional flow area (L^2).

The hydraulic gradient represents the total head loss of the fluid over a given distance. The hydraulic gradient can be evaluated by dropping the velocity head terms from the Bernoulli equation, since the kinetic energy is negligible because the velocity of the flow of water through the soil is very slow. This leaves only the pressure head and the elevation head to supply the driving force (Bucks, 1968). The sum of these two can be written as:

$$\emptyset = p/\rho g - h \qquad (2)$$

where $\emptyset$ is the hydraulic head (L), p is the pressure (F/L^2), ρ is the density of the water (FT^2/L^4), g is the gravitational constant (L/T^2), and h is the elevation head measured from a reference plane (L). This shows that:

$$i = \Delta\emptyset/\Delta l \qquad (3)$$

where $\Delta\emptyset$ is the change in the hydraulic head between two points and Δl is the average flow distance between the two points.

Point methods of evaluating field hydraulic conductivity. Many field test methods have been developed to measure the hydraulic conductivity of soil. Point field procedures have been developed for conditions above and below a water table. Discussion of the point field methods below a water table is covered first.

The auger hole method measures the average horizontal hydraulic conductivity of a soil profile from the water table to a short distance below the bottom of the hole. Diserens (1934) first developed the technique of using auger holes for the measurement of hydraulic conductivity. Hooghoudt (1936) and Ernst (1950) improved and modified the technique and provided graphical solutions. Maasland and

Haskew (1957) described the method in great detail and concluded after thousands of tests in Australia that it accurately measures the hydraulic conductivity of a soil.

A circular hole, usually four inches, is augered and permitted to come to equilibrium. The water is bailed out to flush the side-walls and the water is again permitted to come to equilibrium. Then the water is bailed out and the rate of water elevation rise with time is recorded and used to calculate the hydraulic conductivity.

Childs (1950) proposed the use of the two hole method for non-layered soils. This method uses two auger holes of equal diameter and of an equal depth below the water table preferably to an impermeable layer. Water is pumped out of one hole into the other at a steady and known rate. This creates a small head difference between the two holes. The hydraulic conductivity can be calculated from the head difference, pumping rate, and the geometry of the system.

Kirkham (1955) proposed the use of the four well method to eliminate the effects of surface sealing in the wells where it may exist in Childs two hole method and suggested that two more cased wells of the piezometer type be placed between the two unlined holes. The rate of water movement between the two outer holes and the head difference between them is measured. The ratio of water movement to the difference in head between the two inner wells would then be a measure of the hydraulic conductivity.

Kirkham (1946) first introduced the piezometer method. This method was then developed by Luthin and Kirkham (1949) and Reeve and Kirkham (1951). The piezometer method uses a seamless tube installed in an auger hole 1/16 inch less in diameter than the tube. The hole is augered out six inches at a time and the tube driven to within one inch of the bottom of the hole. This process is repeated until the desired depth is reached. A six inch cavity beyond the end of the tube is then augered out. The hole is flushed and permitted to come to equilibrium. The water is again removed and the rate of rise with time is recorded and used to calculate the hydraulic conductivity value. This method which measures predominately the horizontal conductivity is well suited to measuring the conductivity of stratified soils.

The tube method is essentially the same as the piezometer method except that no cavity is drilled beyond the end of the tube (Frevert and Kirkham, 1948). The system is developed as in the piezometer method and measurements are taken in the same way. The tube method measures the vertical hydraulic conductivity instead of the horizontal as in the piezometer method.

The Pomona well point method is a simple test to perform (American Society of Agricultural Engineers Drainage Committee, 1962).

A King soil tube is driven to the approximate depth at which the measurement is to be made. The soil is removed from the tube and a well point is driven six to eight inches below the end of the tube. The water table is permitted to reach equilibrium and its position is measured. A small diameter suction tube is inserted to a point three inches below the water table. This three inch head difference is maintained and the rate of outflow is measured. This can be converted to the hydraulic conductivity through the use of an empirical equation.

Several methods have also been developed to measure the saturated hydraulic conductivity above the water table under field conditions. The shallow well pump-in method was developed by the Bureau of Reclamation to measure saturated hydraulic conductivity of sites without a water table (Winger, 1960). Measurements are made of the volume of water flowing from an auger hole in which the head of water is held constant. The hydraulic conductivity is a composite rate for the entire depth being tested but generally reflects the more permeable layers. In this method a hole is augered to the desired depth and the rate of water flow into the soil under constant head is measured. From this data the hydraulic conductivity is calculated.

Values of hydraulic conductivity found by this method are generally lower than values found by the auger hole method. The ratio of the hydraulic conductivity values of the pump-in to the auger hole were found to be approximately 0.50 by Talsma (1960) and 0.85 by Winger (1960).

The permeameter method is based on the measurement of the water outflow rate in an auger hole above a water table (Winger, 1960). A large diameter hole is excavated and an 18 inch cylindrical sleeve is placed in the middle. This sleeve penetrates the soil to a known depth below the bottom of the hole. The same water level inside and outside the sleeve is maintained. The water infiltration rate from the inside of the sleeve is measured. The pressure at a known depth below the cylinder is measured through the use of tensiometers. The vertical hydraulic conductivity is calculated by Darcy's law. This method is limited because positive pressures may develop if the soil is underlain by a restricting layer. Although relatively easy to use, this method is time consuming and requires a large amount of specialized equipment.

The double tube method uses two cylinders to determine the hydraulic conductivity of the soil (Bouwer, 1964). An auger hole is excavated to the desired depth, the bottom of the hole is cleaned, and a thin layer of sand is spread on the bottom. Two concentric cylinders with a ratio of diameters of 1.7 or larger are forced into the soil one inch. Both cylinders are filled with water and maintained at the same depth for several hours. The water supply to the center cylinder is cut off and the water levels of the two cylinders are maintained at the same level. The water height versus time is recorded and the data plotted to get a falling water height versus time curve. Next, the water level is restored to the original height. The supply of water to the center cylinder is again cut off and the water height versus time is measured while the water level in the outer cylinder is held constant. This data is plotted to obtain a constant water height versus time curve. The vertical hydraulic conductivity can be calculated using these two curves and an empirical equation.

The pond infiltration method utilizes a large surface area (Luthin, 1966). The area is diked with a ring of soil and filled with water. A circular ring of 400 centimeters is recommended. The procedure is as follows: water is added as needed; when enough water has been added to soak down to the layer of which the conductivity is to be determined, no more water is added. The rate of the falling water level is then measured. Since all flow is due to gravity, the hydraulic gradient is assumed to be unity and the hydraulic conductivity is calculated from Darcy's law.

Drain line methods of evaluating field hydraulic conductivity. Several researchers have evaluated drainage system hydraulic conductivities and compared them with point test method values. They evaluated the drain line values by recording water table elevations and drain line discharges and using these data, the drainage system geometry, and a drainage spacing equation to solve for the hydraulic conductivity.

Hoffman and Schwab (1964) and Perrier, et al. (1972) used two forms of an equation developed by van Schilfgaarde to evaluate the drain line hydraulic conductivity for a silty clay soil in Ohio and California, respectively. Hoffman and Schwab found that the drain line value was approximately one-half of the auger hole value, while Perrier found the drain line value was triple the values found by the piezometer method. Skaggs, et al. (1973) compared drain line hydraulic conductivity values determined from equations developed by van Schilfgaarde, Bouwer and van Schilfgaarde, Glover, and Hammad with hydraulic conductivity values determined from core samples of the North Carolina costal plain. The values found by the drainage design equations were 60, 67, 79, and 22 percent of the core sample value, respectively.

Johnston, et al. (1963) and DeBoer and Johnson (1967) compared drain line hydraulic conductivities from equations developed by Kirkham to auger hole method values for a California rice field and an Iowa floodplain, respectively. Both studies found that the auger hole values provide a good approximation of the drain line hydraulic conductivity.

Lembke (1967) compared auger hole and drain line outflow hydraulic conductivities on a lake plain soil which borders the

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experimental area of this study. The drain line values were determined by a numerical solution using a relaxation technique. Lembke found that the auger hole hydraulic conductivities were about 15 percent smaller than the drain line values. As shown by these studies, the point measurements in the field were not consistently higher or lower than the drain line values.

Drainage Envelope Studies

An envelope is used to facilitate water entry into drain lines, to prevent soil from moving into the drain lines, and to provide a stabilizing foundation for the drain. Gravel and sand have been widely used in envelopes, but many other materials have been used with varying success.

Gravel envelope studies. Karl Terzaghi (1948), in 1921, made the first gravel envelope recommendation:

$$\frac{D_{15} \text{ filter}}{D_{85} \text{ base}} < 4 < \frac{D_{15} \text{ filter}}{D_{15} \text{ base}} \qquad (4)$$

where D_{15} and D_{85} are the particle sizes at which 15 percent and 85 percent of the particle weight is smaller. The terms filter and base refer to the gravel envelope and base material.

Several researchers have since developed design criteria for envelopes. The principal ones are Bertram (1940), United States Army Corps of Engineers (1941), Leatherwood and Peterson (1954), and

United States Department of the Interior (1955). Their proposed design criteria are shown in Table 1.

Table 1. Proposed Design Criteria of Particle Size Ratios for Gravel Envelopes

Researcher	$\frac{D_{15}\text{ filter}}{D_{15}\text{ base}}$	$\frac{D_{50}\text{ filter}}{D_{50}\text{ base}}$	$\frac{D_{15}\text{ filter}}{D_{15}\text{ base}}$
Bertram	9.0		
United States Army Corps of Engineers			5.0 to 10.0
Leatherwood and Peterson		5.3	4.1
United States Department of the Interior		5.0 to 10.0	

In addition the gradation curves of the envelope and base material should be approximately parallel.

Several recommendations have been made about the thickness of envelopes. The United States Department of Agriculture (1960) recommends at least three inches of gravel. The Edward E. Johnson Company (1955) suggests that gravel envelopes have a thickness of three to nine inches. Des Bouvrie (1962) made the following conclusions concerning envelope thickness and the filter-base ratios at the D_{50} grain size:

1. ½ to 1 inch thick for filter-base ratios less than 12
2. 3 inches for filter-base ratios of 12 to 24

3. 6 inches for filter-base ratios of 24 to 28

4. 9 inches for filter-base ratios of 28 to 40

5. 12 inches for filter-base ratios of 40 to 52

Bucks (1968) concluded that a gravel envelope is essential for the protection of a tile drain in the coarse silt base material of the experimental plot of this study. He found that a six inch envelope of pit run gravel will serve this purpose. He also stated that an envelope of any thickness will increase the tile outflow over a line without an envelope. The drain lines without envelopes failed rapidly by filling with soil.

Other envelope materials. Many materials have been studied for use as envelope materials around drain lines. Most researchers, in evaluating the effectiveness of these materials, have compared drain line discharges or visually observed the material after some period of field operation.

Sisson and Jones (1962) used various materials for envelopes around tile lines in a laboratory experiment in a medium fine sand. They tried corn cobs, sawdust, straw, gravel, fiberglass, a fiberglass and vinyl sheet combination, and no envelope. They found that the no envelope case failed quickly and corn cobs were not much better. The remaining envelope materials provided adequate protection against soil movement into the lines.

Ford, et al. (1968) studied envelopes of fiberglass, sawdust, and phosphate furnace slag in Florida citrus groves. They reported that after periods of four to twelve years the sawdust envelopes were greatly reduced in hydraulic conductivity. The fiberglass and slag also had reduced conductivities, but this was caused by an accumulation of iron compounds in the envelopes.

Taylor and Goins (1967) tried envelopes of corn cobs, wheat straw, crushed stone, and vermiculite under humid field conditions. The lines were visually inspected after eight years and only negligible amounts of sediment were found in all lines including the ones without envelopes. They do not recommend the use of envelopes in silty and clay textured soils in a humid region.

DeBoer, et al. (1972) reported the results of an investigation of the envelopes in the experimental area used for this study. The envelopes were a three inch gravel and a combination three inch gravel and fiberglass material (Figure 1). These experimental drain lines were operated and monitored during the growing season for five of six years after installation to aid in the field evaluation of the experimental envelopes. The envelopes operated without problems during the six year period.

Davis, et al. (1972) tried various combinations of three types of drain lines; plastic, clay, and concrete, with three envelope materials; sand, pea gravel, and sand with the fines removed. They also installed observation lines with no envelope, chat, dirty alluvial sand, plastic for velocity control, and a plastic sheet over the top of a line. They found that any envelope material will perform successfully if carefully installed. The lines without envelopes did fail and the ones with velocity control measures had reduced outflow. Willardson, et al. (1973) studied the effect of backfill settling treatments and the use of clean and dirty envelopes on drain discharge. They found that the settling treatments had no effect on drain line discharge. The use of dirty envelope materials did cause a reduction in drain line discharge.

Fiberglass has been studied by several researchers as to its effectiveness as an envelope material. Overholt (1959) conducted a laboratory study comparing a fiberglass wrap with no envelope in a sandy soil. He found almost twice the discharge rate in the fiberglass protected lines. Buras and Pillsbury (1963) tested the lateral movement along a fiberglass sheet and concluded that it would work satisfactorily as an envelope. Nelson (1960) found that fiberglass sheets with random reinforcement worked much better as an envelope than sheets with longitudinal reinforcement.

Shull (1964) tested the hydraulic conductivities of fiberglass mats under various load conditions in a laboratory study and concluded that the mats would improve the hydraulic characteristics of the area near a drain. Shull (1967) also tested the soil filtering properties of these mats. His results showed that no particles larger than very fine sand will move through the mats easily.

PROCEDURE

Description of the Plot Area

The plot chosen for this study is part of the James Valley Research and Extension Center. This plot area contains three sets of pumped drain lines which are spaced 150 feet apart at an eight foot depth as shown in Figure 2. These lines are surrounded by the two experimental envelopes which were installed in 1967. Lines B and C actually consist of three independent parallel lines. The middle line drains a fifty foot border strip on the east and west edge of the plot. The north and south lines drain an area from the middle of the plot to the border area on the west and east sides, respectively. More details are shown in Figure 3.

Topography of the plot is nearly level since it was once part of the lake plain of post-glacial Lake Dakota. A minimal amount of land leveling was performed on the plot in the late 1940's. Classification of the soil is a Beotia silt loam. The soil profile has three distinct regions as detailed in the Bureau of Reclamation logs in Appendix B. They are: from zero to three feet, granular silt loam; from three to seven feet, laminated silt loam; and from seven to eleven feet, unconsolidated coarse silt loam. Although the soil over the entire plot is classified as a Beotia silt loam, the electrical conductivity of the saturation extract of the subsoil from the east side is about twice that of the subsoil in the west part of the

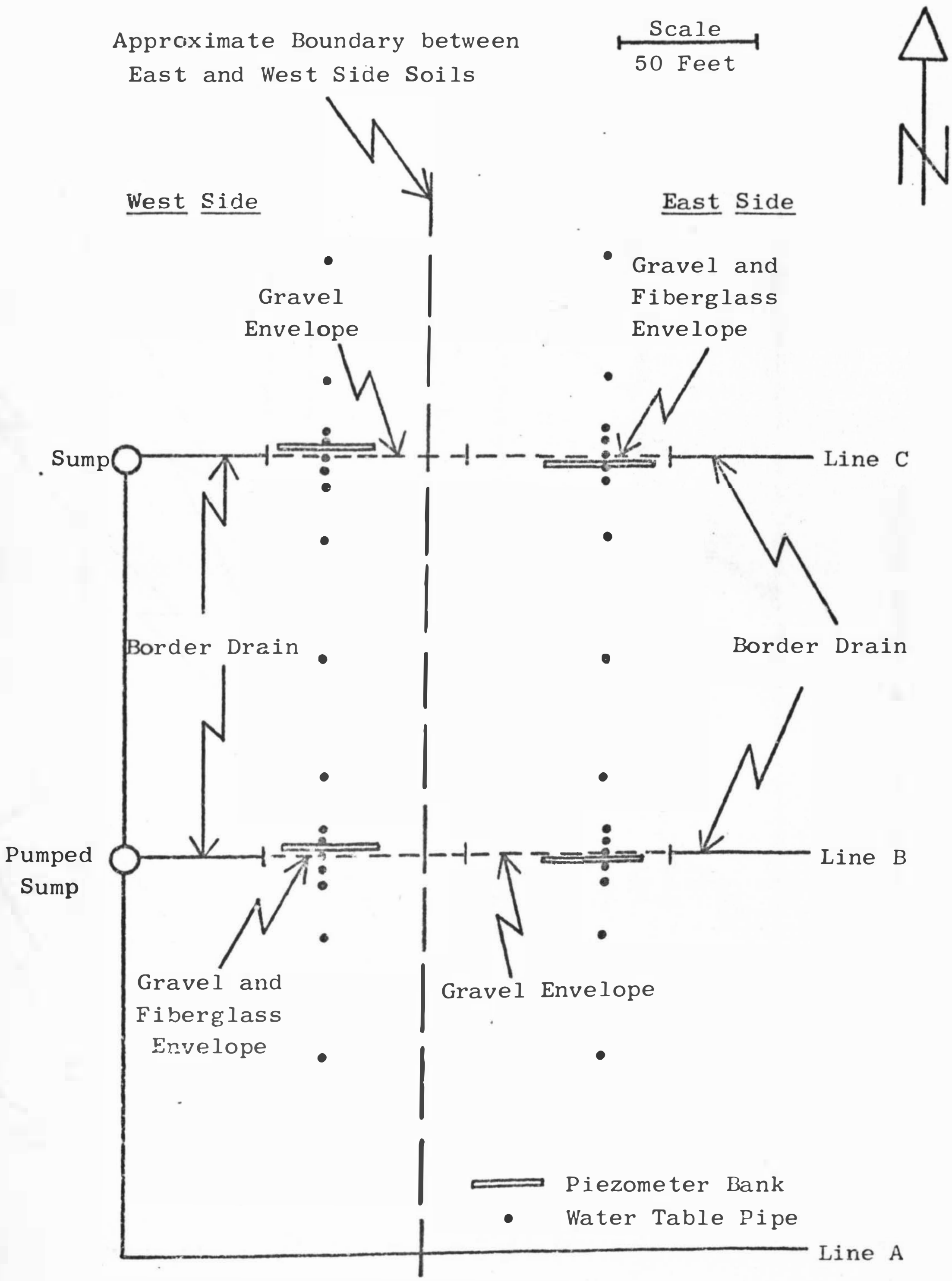

Figure 2. Experimental Plot of James Valley Research and Extension Center

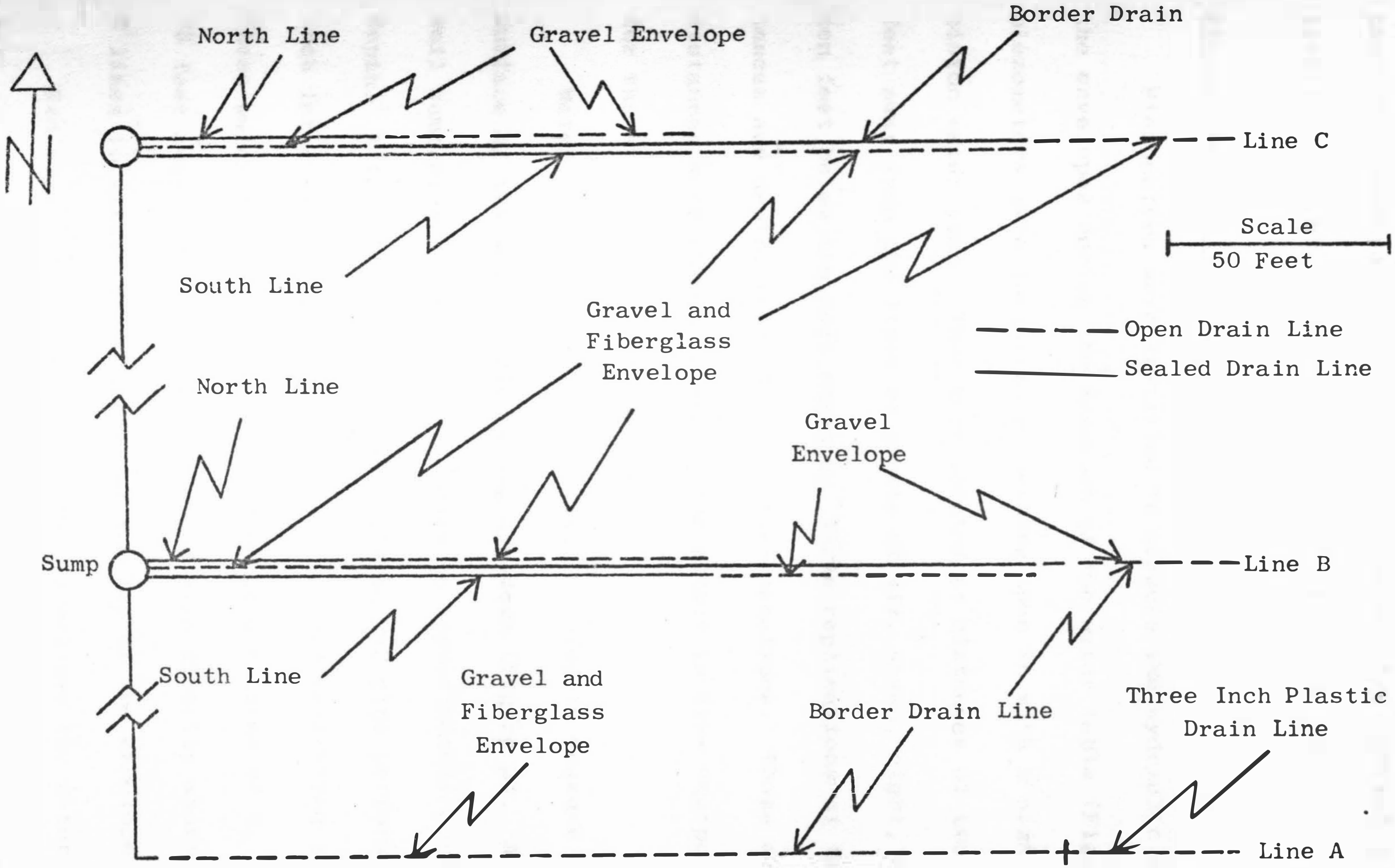

Figure 3. Details of the Experimental Drainage System

plot. From unpublished experimental data (DeBoer, 1973), it appears that this chemical difference may influence the hydraulic conductivity values of the east and west sides of the plot.

Piezometers and Water Table Pipe

Piezometers were installed to measure the hydraulic head around the envelopes during the drawdown of the water table (Figure 2). The piezometers were installed by jetting them in with a high pressure piston water pump. They were located at distances of two and four feet away from the lines at depths of six, seven, eight, nine, and ten feet below the soil surface. Three replications of these distances and depths were placed at four locations. These depths and distances were chosen to enable the author to draw equipotential lines for the water table drawdown period.

Water table pipes were installed and used to measure the phreatic surface of the water table during drawdown (Figure 2). A Giddings soil sampler was used to drill holes to approximately a nine foot depth. A ten foot length of schedule 40 PVC pipe perforated at six inch intervals was placed into the holes and backfilled with sand. These water table pipes were installed at distances of 5, 10, 30, and 75 feet away from each drain line and also directly above the B and C lines to within three inches of the top of the envelope material.

Two blow tubes were constructed to measure the water level in both the piezometers and the water table pipes. These blow tubes consisted of a three foot length of 3/16 inch plexiglass tubing connected to a seven foot length of 5/16 inch Tygon tubing. They were graduated in feet and tenths of feet.

The plot area was irrigated to raise the water table level and left for three days to allow the water table to come to equilibrium. The sump pumps were then started to initiate drain line flow. Water elevation measurements in the piezometers and water table pipes were started when the water level in the sump dropped below the drain line outlets. Measurements were taken continuously for the first three hours and at 9:00 p.m. and 1:00 a.m. the first night. Two sets of measurements were taken each day during the first week and once a day thereafter. Drain line discharges were measured using a bucket and stop watch. These drain line discharge readings were taken continuously for the first four hours and with each set of piezometer readings thereafter.

Hydraulic Conductivity Tests

The saturated hydraulic conductivity of the plot area was evaluated by two point methods; the auger hole and pump-in. The auger hole method evaluates the saturated hydraulic conductivity of a soil when a water table is present, while the pump-in is used in the absence of a water table. The locations of these tests were chosen by dividing the plot into ten foot square units and using a random number table to locate the sites (Figure 4). Eighteen locations for each method were used, nine in the east part and nine in the west part

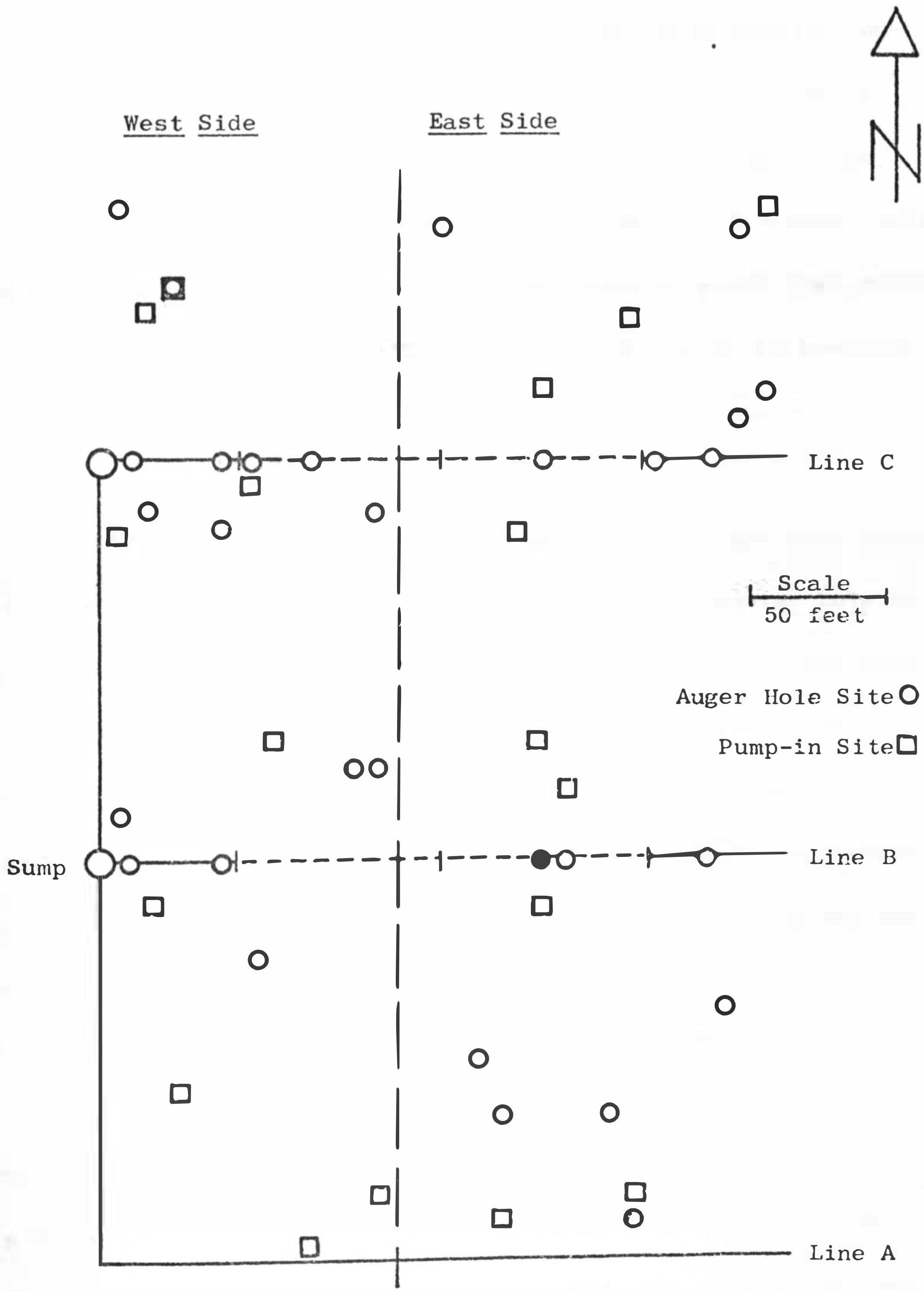

Figure 4. Locations of Pump-in and Auger Hole Test Sites

of the plot. Nine locations were chosen to obtain the most degrees of freedom within the time available for running the hydraulic conductivity tests. The tests were run at two depths; from three to seven feet and from seven to ten feet because of the break in the soil profile at the seven foot depth. Twelve tests by the auger hole method were run on the backfill soil at the three to seven foot depth. These locations were also selected at random. Bureau of Reclamation personnel and equipment aided in the actual running of these hydraulic conductivity tests.

The following procedure was used to conduct the auger hole tests. A mobile drill rig was used to construct a six inch diameter hole to about the water table. A 2.5 inch soil core was taken from the test zone and checked to insure uniformity of the soil. A four inch diameter hole was then hand augered to the desired depth below the water table and a casing made of perforated downspout was inserted to prevent sloughing of the walls. The water level was permitted to come to equilibrium and was bailed out to flush the soil pores. The water was again permitted to come to equilibrium and was bailed out once more. A stopwatch was started at the initiation of the second bailing. A float apparatus was inserted in the auger hole and water levels recorded on a strip chart every 15 seconds (Figure 5). The rate of water rise in the hole was recorded from the charts and used to

Figure 5. Measurement of Water Table Elevations for the Auger Hole Method

Figure 6. Equipment Used for the Pump-in Method

calculate the hydraulic conductivity by the method described by Winger (1960).

The following procedure was used for the pump-in method. The mobile drill was used to drill a hole to the test depth. Soil core samples were taken and checked for soil uniformity. A four inch diameter hole was then hand augered and the sides scratched to remove any sealing effects the augering may have caused. The casing was inserted to prevent the walls from sloughing in. The float apparatus was placed in the casing such that a constant two foot head of water would be maintained. A recorder was used to measure water inflow rate versus time (Figure 6). From these data the hydraulic conductivity of the soil was determined. Pilot holes were also drilled to locate the water table level. The effect of the water table on the rate of inflow was evaluated and used to adjust the hydraulic conductivity values. See Winger (1960) for more details about this method.

The drain line hydraulic conductivity values were found using a drainage spacing equation, drain line discharges, water table elevations, and the system geometry (Figure 7). Van Schilfgaarde's equation (1970) was used to calculate the hydraulic conductivity where

$$K = \frac{f\ S^2}{9t\ d_e} \ln \frac{m_o(2d_e+m_t)}{m_t(2d_e+m_o)} \qquad (5)$$

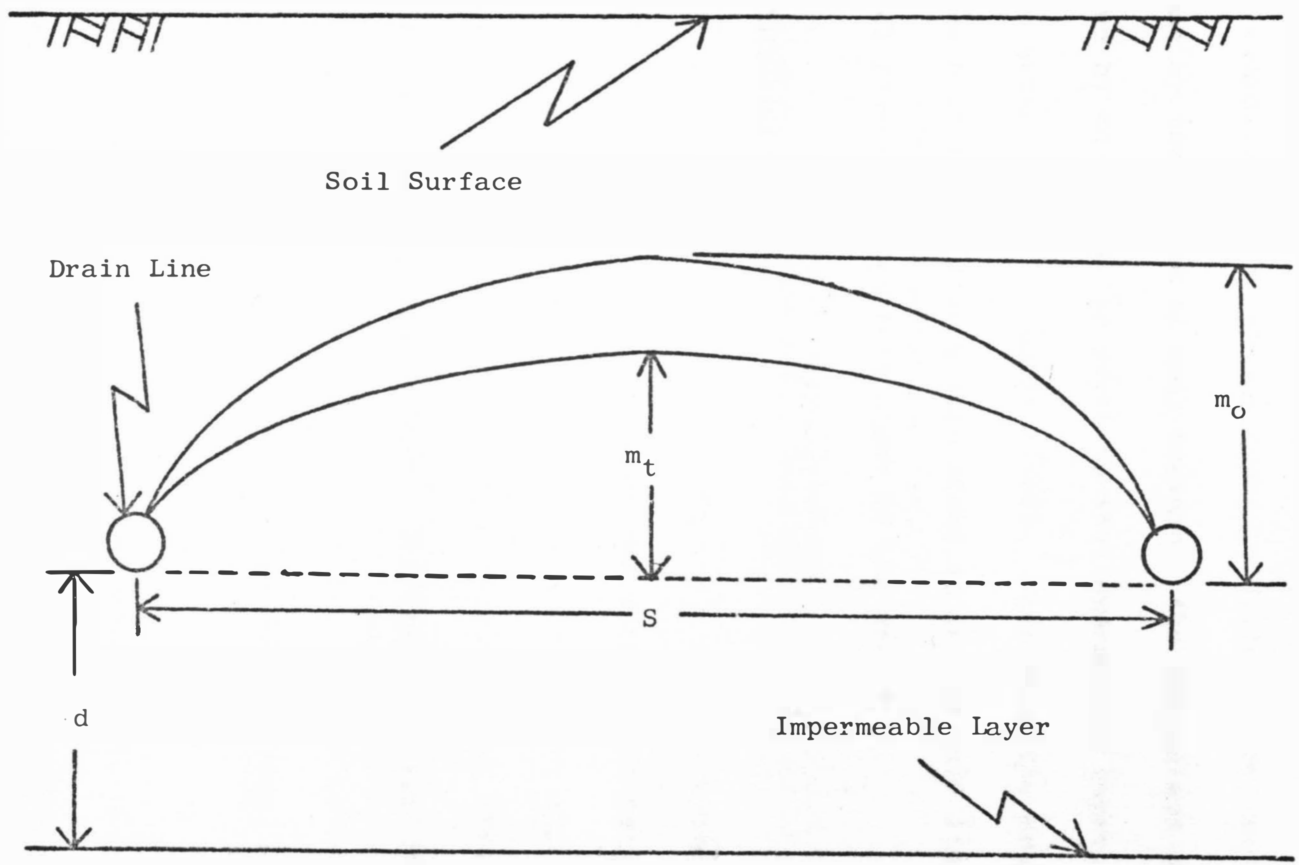

Figure 7. Schematic of the Drainage System Geometry

and K is the hydraulic conductivity, S is the drain line spacing, m_o is the midplane water table height at the start of drawdown, and m_t is the midplane water table height at time t. The drainable porosity is symbolized by f and was found by dividing the water discharge volume by the volume of soil drained. The equivalent depth, d_e, was found by adjusting the depth to the impermeable layer, d, according to graphs found in Luthin (1966). Data from the drain line shown as line B in Figure 2 were used since it is the only line with a symmetrical flow region on both sides of the drain.

Envelope Inspection

Six pits at the locations shown in Figure 8 were excavated for inspection of the envelope materials when the water table dropped below the drain lines. They were excavated by a backhoe to approximately six inches above the lines. A trench three feet deep was also excavated along one side of the drain lines. The remaining soil over the envelope material was carefully removed by hand. Twenty gravel samples from the envelopes around the lines were taken from the top, sides, and bottom of the lines near a drain line joint. Fiberglass samples from below the drain lines were also taken at six joints.

The gravel envelope samples were oven dried for twenty-four hours. These samples were mechanically analyzed by a sieve shaker device to obtain a particle size distribution.

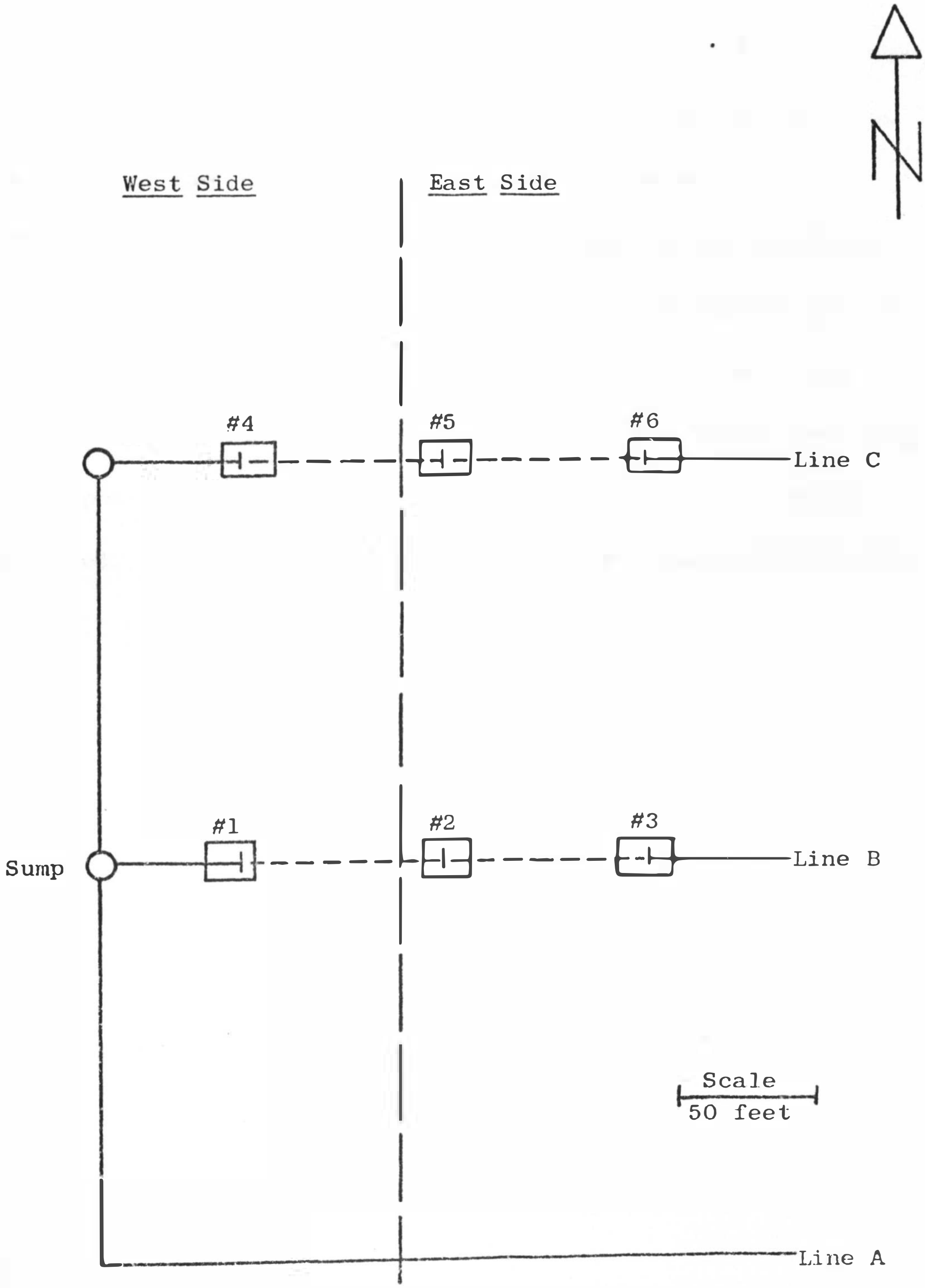

Figure 8. Experimental Plot Area Showing Locations of the Six Envelope Inspection Pits

The hydraulic conductivity of the fiberglass samples was evaluated by use of a constant head permeameter (Black, 1965). Samples with a cross-sectional area of about seven square centimeters were used because of the high hydraulic conductivity of the samples.

The tensile strength of the fiberglass samples was evaluated by directly loading a sample. A narrow strip of fiberglass was cut and clamped on two ends. A container was attached to one clamp with the fiberglass used as a direct supporting member. Weight was added by slowly pouring fine sand into the container until the fiberglass strip failed. The container was weighed and the weight adjusted to a unit width of the fiberglass strip.

RESULTS AND DISCUSSION

The discussion and analysis of the results will be divided into two main units; hydraulic conductivity results and the results of the experimental envelope studies.

Hydraulic Conductivity Results

Field measurements of the saturated soil hydraulic conductivity were made during the summer of 1973 by the pump-in and auger hole methods. Table 2 shows the averages of the auger hole and pump-in test results. It can be noted that the average pump-in values are lower than the average auger hole values and the ratio of the pump-in values to the auger hole values is approximately 0.50 which agrees with the ratio of 0.50 found by Talsma (1960). The hydraulic conductivities were greater for the east side than for the west side.

Table 2. Average Hydraulic Conductivities (ft/day) by the Auger Hole and Pump-in Methods for the East and West Sides of the Experimental Area

Depth (ft)	Side	Pump-in	Auger Hole	Ratio
3-7	East	1.18	2.06	0.57
3-7	West	0.70	1.78	0.39
3-7	Average	0.94	1.92	0.49
7-10	East	1.90		
7-10	West	1.68	3.36 a)	0.50
7-10	Average	1.79		

a) Average of five tests, all the others are averages of nine tests

The auger hole tests at the seven to ten foot depth were not completed because saturated soil conditions could not be maintained. A complete listing of the hydraulic conductivity data is given in Appendix C.

The hydraulic conductivity data were statistically analyzed by an analysis of variance as summarized in Table 3. Two sources of variation had statistically significant F values. The method of measurement was significant at the .01 level and the depth of measurement was significant at the .05 level. This indicates that there is a significant difference between the results of the pump-in and auger hole methods. There is also a significant difference between the results of the three to seven foot and the seven to ten foot regions of the soil profile. The analysis indicates that no statistically significant differences exists between the hydraulic conductivity values of the soils found in the east and west sides of the experimental area even though the average west side values were lower than the average east side values as shown in Table 2. No interactions were statistically significant.

Drain line hydraulic conductivities, shown in Table 4, were evaluated for drain line B over a six year period. The east side of the plot had a smaller average value than the west side. The values do not show a trend with time; therefore it appears that the condition of the drainage system is stable. Drain line hydraulic conductivities were calculated from the drainable porosity values and water table elevations

Table 3. Analysis of Variance for the Pump-in and Auger Hole Methods Hydraulic Conductivity Results

Source	D.F.	Sum of Squares	Mean Square	F
Method	1	289.86	289.86	7.41 **
Side	1	2.70	2.70	0.07
Depth	1	147.64	147.64	4.06 *
Method x Side	1	2.76	2.76	0.08
Method x Depth	1	67.83	67.83	1.86
Side x Depth	1	11.75	11.75	0.32
Error	52	1893.07	36.41	
Total	58			

** Significant at the .01 level
 * Significant at the .05 level

using van Schilfgaarde's equation and the method outlined in the procedure. The validity of the drain line hydraulic conductivity value is dependent on the reliability of the drainable porosity values. The surface of the water table was usually from three to six feet below the soil surface while the drain line values were being evaluated. Water tables were not observed directly over the drain lines after about four hours from the initiation of drawdown during the 1973 data collection period.

Table 5 shows the analysis of variance for the drain line hydraulic conductivity data. There is a statistically significant difference between the results for the east and west sides of the plot area. This is in contrast to results of the pump-in and auger hole tests which did not show a statistically significant difference between

Table 4. Drain Line Hydraulic Conductivities (ft/day) and Drainable Porosities for the Three to Ten Foot Soil Profile Region.

Date	East Side		West Side	
	Drainable Porosity	Hydraulic Conductivity	Drainable Porosity	Hydraulic Conductivity
July 1968	0.030	0.89	0.037	0.86
August 1968	0.030	0.94	0.033	1.26
August 1968	0.029	0.85	0.045	0.84
August 1970	0.018	0.84	0.025	1.20
September 1971	0.035	0.93	0.041	1.09
October 1972	0.026	0.79	0.035	1.21
August 1973	0.016	0.77	0.032	1.10
Average	0.026	0.86	0.035	1.08

the results for the east and west sides. Since the west drain line values represent the west soil and the fiberglass-gravel envelope combination and the east values represent the east soil and the all gravel envelope combination, it is not possible to make any statistical conclusions regarding the influence of the two envelopes on drain line hydraulic conductivities. However, if it is assumed there is no difference in the east and west soil as indicated by the analysis of variance in Table 3, then the difference in the drain line values can be attributed to the different envelopes. Therefore the combination gravel and fiberglass envelope appears to have better hydraulic characteristics than the all gravel envelope.

Table 5. Analysis of Variance for the Drain Line Hydraulic Conductivity Values

Source	D.F.	Sum of Squares	Mean Square	F
Sides	1	0.1716	0.1716	10.53**
Error	12	0.1959	0.163	
Total	13			

** Significant at the .01 level

Table 6 shows a comparison among the average hydraulic conductivities by the auger hole, pump-in, and drain line methods for the three to ten foot depth. The auger hole value was the largest and the drain line value the smallest. The auger hole and pump-in values were 260 and 140 percent of the drain line value, respectively. The pump-in value represents the average of 36 tests and the auger hole value represents the average of 23 tests. The drain line value is the average of the data shown in Table 4.

Table 6. Average Hydraulic Conductivities (ft/day) for the Pump-in, Auger Hole, and Drain Line Methods

Depth (ft)	Pump-in	Auger Hole	Drain Line
3-10	1.38	2.64	0.97

The auger hole method was also used on the backfill soil to assess the possible difference between the hydraulic conductivity of the undisturbed and disturbed (backfill) soils. The data from these tests are shown in Table 7.

Table 7. Hydraulic Conductivity Values (ft/day) for the Auger Hole Method at the Three to Seven Foot Depth

East Side			West Side	
Backfill	Undisturbed		Backfill	Undisturbed
0.09	1.42		0.25	2.76
1.98	5.12		0.07	0.41
0.56	2.44		0.85	3.06
4.82	1.07		0.04	0.33
0.50	1.92		0.46	1.23
0.61	1.92		1.18	1.73
	3.04			3.20
	0.55			1.35
	1.00			1.95
1.43	2.06	Average	0.47	1.88

The average hydraulic conductivity for both backfill soils is 0.95 as compared to 1.97 for both undisturbed soils. One of the east side backfill values is 4.82 feet per day. This value caused a large increase in the average east side backfill value and in the overall backfill average. The analysis of variance for these data is shown in Table 8. The term, condition, refers to the condition of the soil, backfill or undisturbed. None of the sources of variation were shown to be statistically significant. Therefore it can be assumed that the backfill soil is not different from the undisturbed soil as is assumed in the development of most drainage design equations.

Table 8. Analysis of Variance for the Hydraulic Conductivity Results on the Backfill and Undisturbed Soils by the Auger Hole Method

Source	D.F.	Sum of Squares	Mean Square	F
Side	1	38.31	38.31	1.05
Condition	1	44.38	44.38	1.22
Side x Condition	1	9.86	9.86	0.27
Error	26	945.86	36.38	
Total	29			

Experimental Envelope Results

The two experimental envelopes were investigated by two different methods. Piezometers were used to measure the hydraulic head of the water in the soil surrounding the envelopes during drawdown to detect possible flow resistances caused by the envelope material. The second method was to visually observe the envelopes in their natural state and collect envelope samples for laboratory analyses.

Piezometer data collected during drawdown of the water table was used to determine the hydraulic heads (Appendix D) in the undisturbed soil adjacent to the drain lines. These hydraulic head values used the bottom of the drain as the reference plane which had a hydraulic head of zero. If the envelope material was restricting flow, this would cause an increase in the hydraulic head of water surrounding the drain line. It was assumed that by measuring these possible differences in hydraulic head that flow resistances caused by the envelope materials would be detected.

The piezometric data were transformed into a dimensionless ratio (Appendix E) where a zero value represents a zero hydraulic head at the elevation of the drain line and a one value represents the hydraulic head at the initiation of drawdown. The hydraulic head ratio was used instead of the actual hydraulic head because the initial hydraulic head was not a constant for all drain lines. Hence the ratio was used to provide a common basis for data analysis. The average initial hydraulic heads for the SW, NW, SE, and NE piezometer banks were 3.68, 3.33, 3.56, and 2.81 feet of water, respectively.

Figure 9 shows the time periods during which piezometer measurements were made. Each set of piezometric measurements, which includes one reading of each piezometer, is represented by a shaded rectangle. During the early part of the drawdown, two people were making measurements which accounts for the overlap of measurement set one, two, and three. The objective of the experiment was to collect hydraulic head data that could be statistically analyzed to detect differences in the hydraulic heads in the soil surrounding the experimental envelopes during drawdown. Each piezometric measurement is related to time; therefore it was desirable to eliminate time as a variable to facilitate analysis of the data.

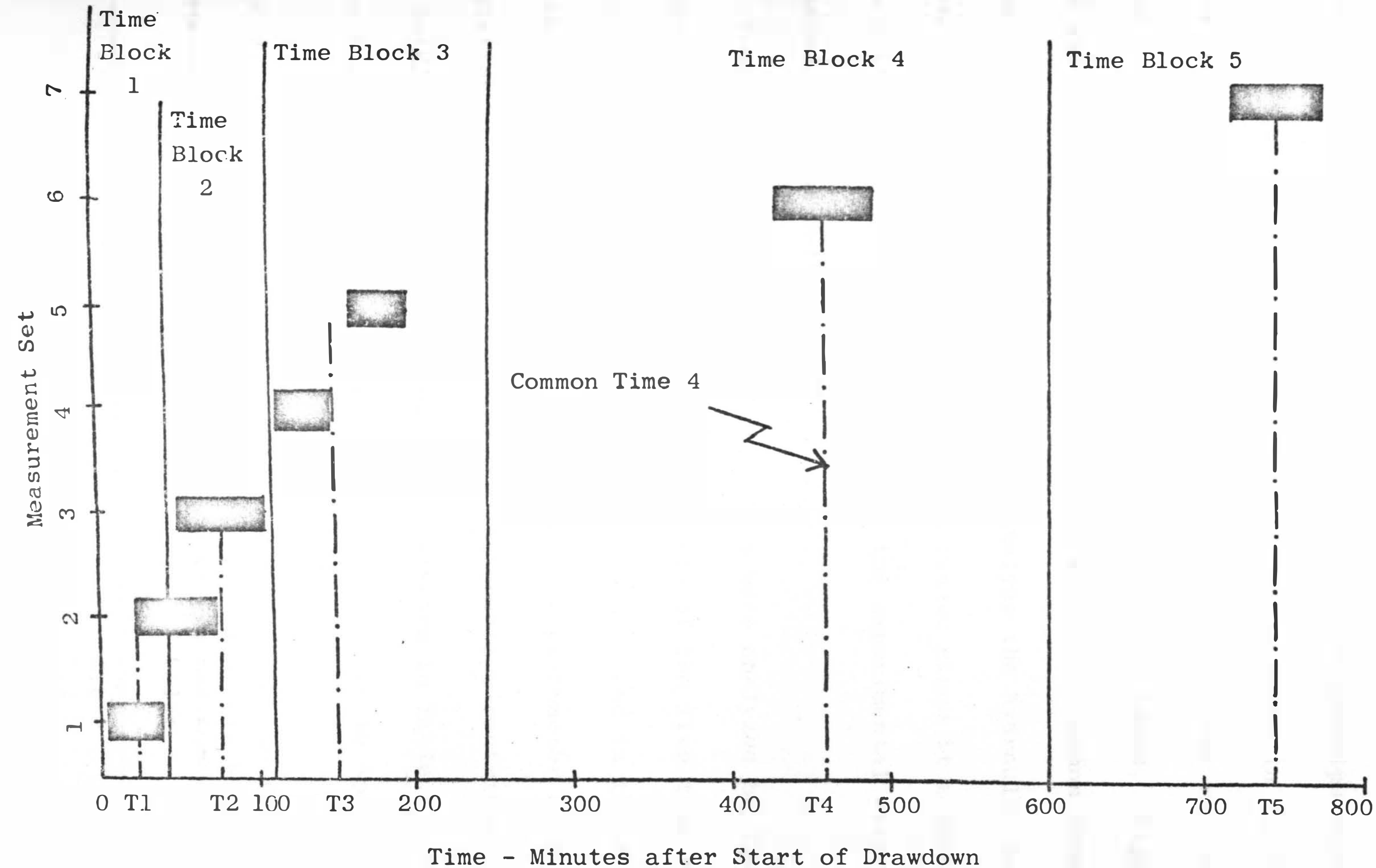

Figure 9. Time Periods for the Piezometer Measurements

The basic procedure used to remove the effect of time was to develop least squares polynomial regression relationships that were used to adjust the data to five common times as shown in Figure 10. Appendix F contains the polynomial regression equations and Appendix G shows actual plots of the data and the regression lines. Figure 9 also shows the time intervals associated with each common time. The overall statistical procedure used to analyze the hydraulic head ratio data is similar in concept to covariance since it adjusts the data with respect to time and preserves the experimental error associated with the data.

These adjusted hydraulic head ratios were analyzed by five separate analyses of variance, one for each of the five time blocks. The data from the six foot piezometers were not used in these analyses because the water level dropped below the piezometer soon after initiation of drawdown. F values for the envelope contribution to the total variation were computed and are shown in Table 9. A complete listing of the analyses of variance is given in Appendix H.

Table 9. F Values for the Envelope Contribution to the Total Variation for the Five Blocks of Piezometric Data

Time (minutes)	F Value
21	6.6 *
76	2.9
150	2.4
460	1.2
745	1.0

* Significant at the .05 level

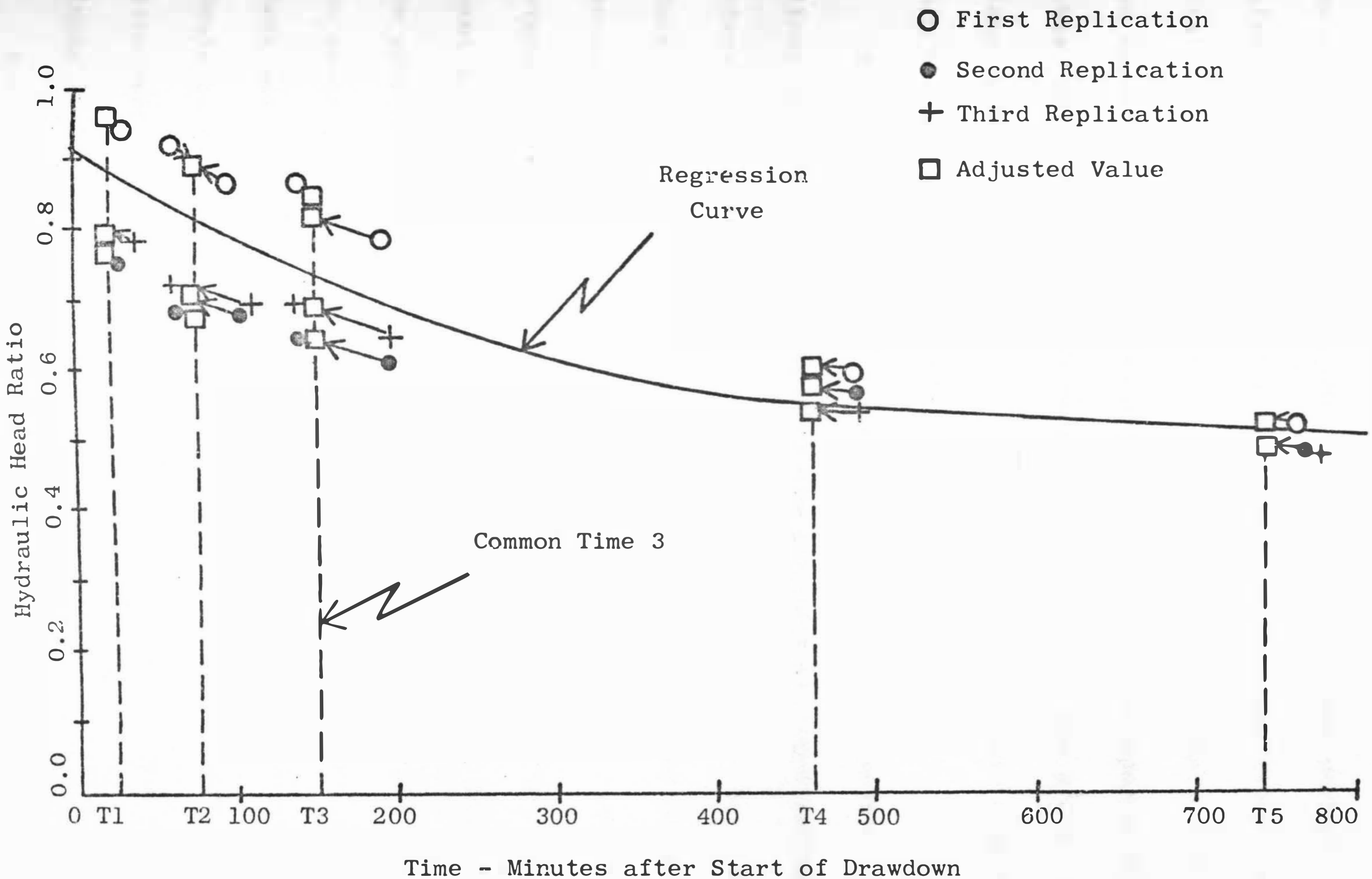

Figure 10. Illustration of Hydraulic Head Ratio Adjustment Procedure

The F value for the envelope component of the total variation was statistically significant during the first time period. The later time periods did not have statistically significant F values. This indicates a difference in the hydraulic characteristics of the two experimental envelopes during the initial development of a water table, but this difference decreased with time. The gravel and fiberglass combination envelope usually had lower hydraulic head ratios than the all gravel envelope.

Table 10 shows the F values for the other main source of variation; soil difference, depth of the piezometers, replication, and lateral distance away from the drain line. It should be noted that there is not a general trend in these F values for any of these sources of variation. Soil difference, which is the difference between the soil on the east and west sides of the plot, was significant for the first three time blocks. The hydraulic head ratios for the west soil were usually lower than the values for the east soil. The depth was significant for all five time blocks thus indicating there was a hydraulic head difference due to piezometer depth which should be expected. The replication and distance sources of variation were not statistically significant for any of the five time blocks.

Six pits were excavated to the drain lines for the inspection of the envelopes. The thickness of the gravel around the lines varied

Table 10. F Values for the Soil, Depth, Replication, and Distance Sources of Variation for the Five Time Blocks

Time (minutes)	Source			
	Soil	Depth	Replication	Distance
21	5.7 *	9.7 ***	1.4	0.4
76	7.7 ***	31.9 ***	0.4	0.5
150	11.6 ***	17.4 ***	1.6	0.7
460	1.9	3.9 *	0.6	1.2
745	2.5	3.6 *	0.6	0.9

*** Significant at the .005 level
** Significant at the .01 level
* Significant at the .05 level

from about one inch to more than three inches. The gravel material appeared to be cohesive because it did not fall away from the sides and bottom of the drain lines. The gravel did not seem to be encrusted on the lines; however minor encrustations were found on one joint out of the 17 inspected. The fiberglass sheet appeared unbroken and in good condition for the entire length observed. It was inspected in the field under a 10x magnifying glass and soil was not observed inside of the fiberglass sheet but only on the edges. The fiberglass was more discolored with soil particles around the joints than between the joints. Overall, the envelope materials appeared to be in good condition and were providing adequate protection for the drain lines. Movement of the soil and envelope material into the drain lines was not noted.

Twenty gravel samples were collected during the field inspection and were tested in the laboratory for particle size distribution. The results of these mechanical analyses are shown in Figure 11 and Appendix I. The curves for the top, side, and bottom are the average of twelve, five, and three samples, respectively. All of the particle size distributions varied considerably among the samples. The bottom curve had the coarsest particle size distribution while the side had the finest particle size distribution.

Figure 12 shows the average gradation curve of the twenty samples as compared to the original envelope gradation curve as shown by Bucks (1968). It should be noted that the sampled gravel material was coarser than the original gravel envelope for the upper 95 percent of the curve. The lowest five percent of the curve shows that there has been inflow and entrapment of the base material within the drainage envelope. The field sample curve shows a larger percentage of fine particles than the original curve does.

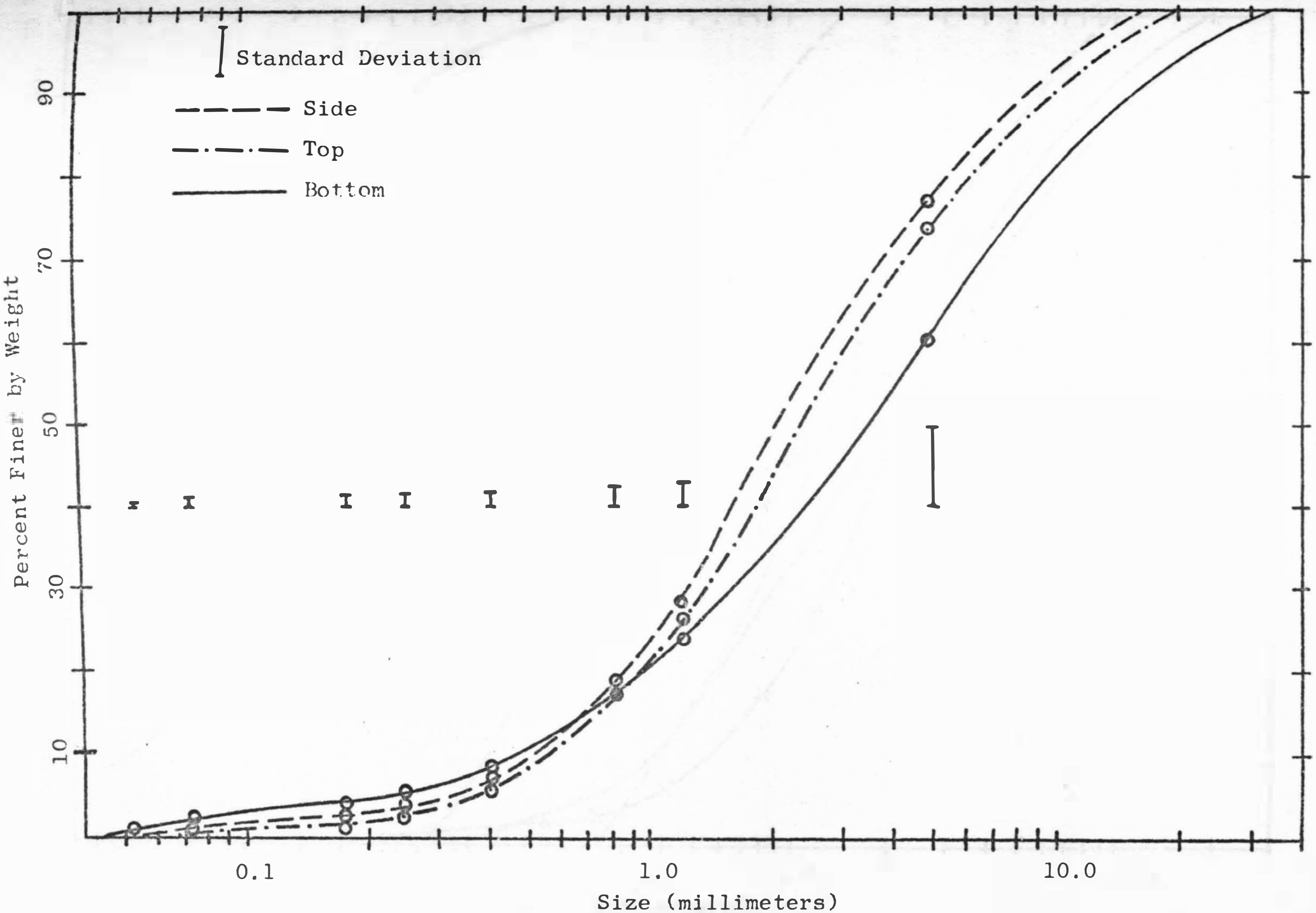

Figure 11. Particle Size Distribution for the Top, Side and Bottom Field Gravel Envelope Samples

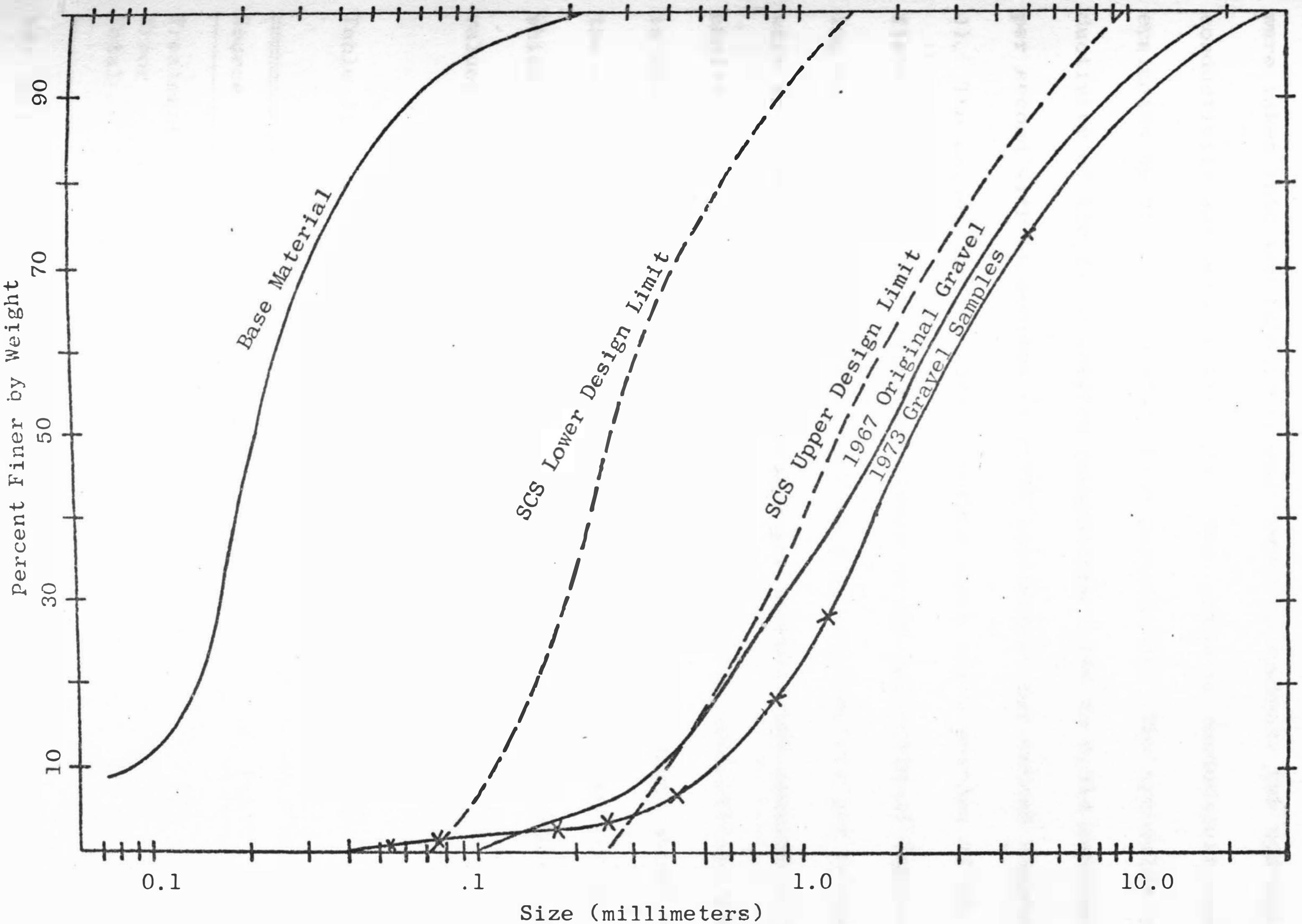

Figure 12. Particle Size Distribution for the Base Material, Original Gravel Envelope Material, and the 1973 Gravel Envelope Samples

Six fiberglass samples taken from underneath the drain lines were taken into the laboratory and tested to evaluate the hydraulic conductivity and tensile strength. The hydraulic conductivity was evaluated by use of a constant head permeameter. The hydraulic conductivity of the field samples ranged from 0.109 to 0.514 centimeters per second with an average of 0.277 centimeters per second (Appendix J). The values for the control sample which was a section of the field envelope fiberglass sheet stored in the Agricultural Engineering Building since 1967 were 0.315 to 0.632 centimeters per second with an average of 0.431 centimeters per second. The results of the analysis of variance for the fiberglass hydraulic conductivity data is shown in Table 11. The treatment source of variation, which is the used fiberglass sheet and the control sheet, was significant, which indicates there is a difference in the hydraulic conductivity values for these two treatments.

Table 11. Analysis of Variance on the Fiberglass Hydraulic Conductivity Data

Source	D.F.	Sum of Squares	Mean Square	F
Treatment	1	0.0943	0.0943	9.33***
Error	30	0.3031	0.0101	
Total	31			

*** Significant at the .005 level

During the collection of the field fiberglass samples it was noted that the fiberglass seemed to rip very easily. For this reason the tensile strength of the samples were checked and compared with the tensile strength of an unused sample. The field samples had a tensile strength ranging from 100.0 to 919.8 grams per centimeter of width with an average of 479.1 grams/cm. The unused fiberglass had a tensile strength in the range of 612.8 to 917.5 grams/cm. and an average of 783.2 grams/cm. The field samples had an average tensile strength of 61 percent of the control fiberglass sample. Table 12 shows the results of an analysis of variance on this tensile strength data (Appendix J). The treatment source of variation was significant at the .01 level, which indicates a statistically significant difference between the tensile strength of the field and control (unused) fiberglass samples.

Table 12. Analysis of Variance for the Tensile Strength of the Fiberglass

Source	D.F.	Sum of Squares	Mean Square	F
Treatment	1	382671.05	382671.05	8.45 **
Error	27	1223215.98	45304.30	
Total	28			

** Significant at the .01 level

SUMMARY AND CONCLUSIONS

Summary

The saturated hydraulic conductivity of the soil within an experimental plot area at the James Valley Research and Extension Center was measured during the summer of 1973 by the pump-in and auger hole methods. Drain line hydraulic conductivities were evaluated from a transient drainage equation, drain line discharges, and the drainage system geometry. The auger hole method produced the largest average hydraulic conductivity value of 2.64 feet per day for the three to ten foot depth. The pump-in average was 1.38 feet per day. The average drain line hydraulic conductivity was 0.97 feet per day.

The hydraulic conductivity of the trench backfill soil on the experimental plot was evaluated by the auger hole method and compared with the undisturbed soil hydraulic conductivity. No statistical difference was found between the undisturbed and backfill soils.

The hydraulic characteristics of two experimental envelopes; a three inch all gravel envelope and a combination three inch gravel and fiberglass envelope, installed within the plot area in 1967 were also studied. Piezometers were installed and used to measure the hydraulic head in the soil surrounding the experimental envelopes. These hydraulic heads were measured during drawdown of the water table and were analyzed to compare the relative hydraulic characteristics of the envelopes. These data indicated a statistically significant difference between the envelopes during the initial development of the water table. However, there was no statistically significant difference between the envelopes for a fully developed water table. Also, no water was observed standing over the drain lines after approximately four hours from the initiation of draw-down.

Pits were excavated to the drain lines and the experimental envelopes were visually observed at seventeen joints. The gravel materials appeared to be cohesive and varied in thickness from one inch to more than three inches at the drain line joints. All the envelope materials appeared to be in good condition and were providing adequate protection for the drain lines. Six samples of the fiberglass and twenty of the gravel were taken from the envelopes and analyzed in the laboratory. The gravel samples were mechanically analyzed for particle size distribution. They were found to be coarser than the original gravel envelope material but some inflow of the silt base material was noted. The fiberglass samples were tested for hydraulic conductivity and tensile strength and these values were compared to the values found by testing a control (unused) piece of fiberglass material. The hydraulic conductivity and tensile strength were approximately 60 percent of the control piece of fiberglass.

Conclusions

The following conclusions were reached as a result of this study:

1. Both the auger hole and pump-in methods gave average hydraulic conductivity values larger than the average drain line hydraulic conductivity value. They were 260 percent and 140 percent of the drain line average, respectively.

2. The pump-in method gave a more accurate estimation of the hydraulic conductivity value for the drainage design equations than the auger hole method for this experimental soil.

3. The ratios of the pump-in hydraulic conductivity values to the auger hole values were approximately 0.50.

4. There was no statistically significant difference between the auger hole hydraulic conductivity values for the undisturbed and disturbed (backfill) soils.

5. No statistical difference was detected between the hydraulic conductivity values of the chemically different soils in the east and west sides of the experimental area.

6. Both of the experimental envelopes; the three inch all gravel envelope and the combination three inch gravel and fiberglass envelope, gave satisfactory protection against siltation of the drain lines during the six year experimental period.

7. There was a statistically significant difference in the piezometric head distribution near the two experimental envelopes during the initial development of the water table. However, no statistically significant difference was found for the fully developed water table situation.

8. There was a reduction in both the hydraulic conductivity and tensile strength of the six year old field fiberglass material to approximately 60 percent of the original values.

REFERENCES

1. American Society of Agricultural Engineers Drainage Committee. 1962. Measuring Saturated Hydraulic Conductivity of Soils. Special Publication SP-SW-0262. ASAE.

2. Bertram, G. E. 1940. An Experimental Investigation of Protective Filters. Soil Mech. Ser. No. 7. Harvard Graduate School Eng., Cambridge, Mass.

3. Black, C. A. 1965. Methods of Soil Analysis, Part 1. American Society of Agronomy No 9. Madison, Wisconsin.

4. Bouwer, Herman. 1964. Measuring Horizontal and Vertical Hydraulic Conductivity of Soil with the Double-Tube Method. Soil Science Society of America Proceedings, Vol. 28.

5. Bucks, D. A. 1968. A Gravel Envelope for a Tile Drain in a Coarse Silt Base Material, Unpublished M. S. Thesis, South Dakota State University: Brookings, South Dakota.

6. Buras, N. and Pillsbury, A. F. 1963. Lateral Movement of Water Along a Glass Fiber Sheet Used as a Draintile Filter. Trans. ASAE 6(2):173,176.

7. Childs, E. C. and Collis-George, N. 1950. The Permeability of Porous. Proc. Roy. Soc. (London), A201:392-405.

8. Darcy, Henry. 1856. "Les Fontaines Publique de la Ville de Dijon." Dalmont, Paris. Original not available; cited in Hillel, D. 1971. Soil and Water: Physical Principles and Processes. Academic Press. New York and London.

9. Davis, S., Willardson, L. S., Spencer, J. R., and Mulder, D. 1972. Drain Envelope Performance in a Sandy Soil. National Drainage Symposium Proceedings, pp. 8-10, 19. ASAE. St. Joseph, Michigan.

10. DeBoer, D. W. and Johnson, H. P. 1967. Floodplain Drainage Investigations of Big Wyacondah Watershed, Davis County, Iowa. Unpublished Report, Department of Agricultural Engineering, Iowa State University.

11. DeBoer, D. W., Bucks, D. A., Lembke, W. D., and Wiersma, J. L. 1972. Envelope Performance in a Coarse-Silt Base Material. National Drainage Symposium Proceedings, pp. 5-7. ASAE. St. Joseph, Michigan.

12. DeBoer, D. W. 1973. Unpublished Hydraulic Conductivity Data. South Dakota Agricultural Experiment Station Project No. 658. Department of Agricultural Engineering, South Dakota State University: Brookings, South Dakota.

13. Des Bouvrie, C. 1962. Laboratory Investigations at Design Criteria for Drain Tile Filters, Unpublished M.S. Thesis, Colorado State University: Fort Collins, Colorado.

14. Diserens, E. 1934. Beitrag zur Bestimmung der Durchlassigkeit des Bodens in Naturlicher Bodenlagerung. Schweiz. Landw. Monatshefte. 12:188-198, 204-212. Original not available; cited in Black, C. A. 1965. Methods of Soil Analysis, Part 1. American Society of Agronomy No. 9. Madison, Wisconsin.

15. Dumm, L. D. 1954. Drain-Spacing Formula. Agr. Eng. 35(10): 726-730.

16. Ernst, L. F. 1950. Een nieuwe formule voor the berekening van de doorlaatfactor met de boorgatenmethode. Rap. Landbow-proefsta. en Bodemkundig Inst. T. N. O. Groningen, The Netherlands (mimeo). Original not available; cited in Black, C. A. 1965. Methods of Soil Analysis, Part 1. American Society of Agronomy, No. 9. Madison, Wisconsin.

17. Ford, H. W., Calvert, D. V., and Beville, B. C. 1968. Hydraulic Conductivity of Soils and Filter Materials in Florida Wetland. Citrus. Trans. ASAE 11(4):566-567.

18. Frevert, R. K. and Kirkham, Don. 1948. A Field Method for Measuring the Permeability of Soil Below a Water Table. Proc. Highway Res. Board 28:433-442.

19. Hillel, Daniel. 1971. Soil and Water: Physical Principles and Processes. Academic Press, New York and London.

20. Hoffman, G. J. and Schwab, G. O. 1964. Tile Spacing Prediction Based on Drain Outflow. Trans. ASAE 7(4):444-447.

21. Hooghoudt, S. D. 1936. Bijdragen tot de Kennis van Ennige Natuurkundige Grootheden van den Grond, 4. Versl. Landb., Ond. 42(13)B:449-541. Original not available; cited in Black, C. A. 1965. Methods of Soil Analysis, Part 1. American Society of Agronomy, No. 9. Madison, Wisconsin.

22. Johnson, E. E., Inc. 1955. Judging Proper Gravel Pack Thickness. Johnson National Driller Journal. 27(2):1-4.

23. Johnston, W. R., Letey, J., and Pillsbury, A. F. 1963. Hydraulic Conductivity Values Compared Using (1) Ponded Water Theory and (2) an Auger Hole Method. Trans. ASAE 6(3):249-250.

24. Kirkham, Don. 1946. Proposed Method for Field Measurement of Permeability of Soil Below the Water Table. Soil Sci. Soc. Amer. Proc. 10:58-68.

25. Kirkham, Don. 1955. Measurement of the Hydraulic Conductivity of Soil in Place. Symposium on Permeability of Soils. Amer. Soc. Test. Mat. Spec. Tech. Pub. 163:80-97.

26. Leatherwood, F. N. and Peterson, D. F. 1954. Hydraulic Head Loss at the Interface between Uniform Sands of Different Sizes. Trans. Amer. Geophys. Union 35(4):588-594.

27. Lembke, W. D. 1967. Observed and Predicted Tile Flow on a Lake Plain Soil. Trans. ASAE 10(1):142-144.

28. Luthin, J. N. 1957. Drainage of Agricultural Lands. American Society of Agronomy No. 7. Madison, Wisconsin.

29. Luthin, J. N. 1966. Drainage Engineering. John Wiley and Sons, Inc. New York, New York.

30. Luthin, J. N. and Kirkham, Don. 1949. A Piezometer Method for Measuring Permeability of Soil in situ Below a Water Table. Soil Sci. 68:349-358.

31. Maasland, M. and Haskew, H. C. 1957. The Auger Hole Method of Measuring the Hydraulic Conductivity of Soil and its Application to the Drainage Design. Proc. 3rd Intern. Congr. Irrigation and Drainage. 8:69-114.

32. Nelson, R. W. 1960. Fiberglass as a Filter for Closed Tile Drains. Agr. Eng. 41(10):690-693.

33. Overholt, Virgil. 1959. Fiber Glass Filters for Tile Drains. Agr. Eng. 40(10):604-607.

34. Perrier, E. R., MacKenzie, A. J., Grass, L. B., and Shull, H. H. 1972. Performance of a Tile Drainage System: An Evaluation of a Tile Design and Management. Trans. ASAE 15(3):440-444.

35. Reeve, R. C. and Kirkham, Don. 1951. Soil Anisotrophy and Some Field Methods for Measuring Permeability. Trans. Amer. Geophys. Union 32:582-590.

36. Schwab, G. O., Frevert, R. K., Edminster, T. W., and Barnes, K. K. 1966. Soil and Water Conservation Engineering. John Wiley and Sons, Inc. New York, New York.

37. Shull, H. H. 1964. Hydraulic Characteristics of Glass-Fiber Materials. Trans. ASAE 7(2):120-122.

38. Shull, H. H. 1967. Soil-Filtering Properties of Glass-Fiber Mat. Trans. ASAE 10(5):655-657.

39. Sisson, D. R. and Jones, B. A. 1962. Filter Materials for Tile Drains in a Medium Sand—A Laboratory Comparison. Trans. ASAE 5(1):54-58.

40. Skaggs, R. W., Kriz, G. J., and Bernal, R. 1973. Field Evaluation of Transient Drain Spacing Equations. Trans. ASAE 16(3):590-595.

41. Talsma, T. 1960. Comparison of Field Methods of Measuring Hydraulic Conductivity. Trans. Congr. Irrigation and Drainage IV(6):C145-C156.

42. Taylor, G. S. and Goins, T. 1967. Field Evaluation of Tile Drain Filters in a Humid Region Soil. Research Circular 154. Ohio Agricultural Research and Development Center. Wooster, Ohio.

43. Terzaghi, K. and Peck, R. B. 1948. Soil Mechanics in Engineering Practice. John Wiley and Sons, Inc. New York, New York.

44. United States Army Corps of Engineers. 1941. Waterways Experiment Station. Investigation of Filter Requirements for Underdrains. Technical Memorandum No. 183-1.

45. United States Department of Agriculture. 1960. Soil Conservation Service. Tile Appurtenances. Drainage, National Engineers Handbook Section 16, Chapter 5. Washington, D. C.

46. United States Department of the Interior. 1955. Bureau of Reclamation. The Use of Laboratory Tests to Develop Design Criteria for Protective Filters. Earth Laboratory Report No. EM-425. United States Department of the Interior, Washington, D. C.

47. United States Department of the Interior. 1965. Bureau of Reclamation. Report on Oahe Unit, Appendix F - Drainage. Missouri-Oahe Project Office, Huron, South Dakota.

48. van Schilfgaarde, J. 1970. Theory of Flow to Drains. Advances in Hydroscience. 6(5):43-106. Edited by V. T. Chow. Academic Press, New York, New York.

49. Willardson, L. S., Davis, S., Mulder, D., and Spencer, J. R. 1973. Drain Envelope Response to Field Treatments. ASAE Paper No. 73-2513.

50. Winger, R. J. 1960. In-Place Permeability Tests and Their Use in Subsurface Drainage. Treatise prepared for the International Commission on Irrigation and Drainage - Fourth Congress - Madrid, Spain. Office of Drainage and Ground Water Engineering, Commissioner's Office, Bureau of Reclamation, Denver, Colorado.

APPENDICES

APPENDIX A

LIST OF SYMBOLS

LIST OF SYMBOLS

A - cross-sectional flow area, L^2

d - depth of impermeable layer below drain lines, L

d_e - equivalent depth of impermeable layer below drain lines, L

D_{15}
D_{50} - particle size at which 15%, 50%, 85% of the particle weight
D_{85} is smaller, L

f - drainable porosity, L^3/L^3

g - gravitational constant, L/T^2

h - elevation head, L

i - hydraulic gradient, L/L

K - hydraulic conductivity, L/T

l - distance, L

m_o - original midplane water height above the drain line, L

m_t - midplane water table height at time t, L

p - pressure, F/L^2

Q - volume flow rate, L^3/T

S - spacing between the drain lines, L

t - time, T

$\emptyset$ - hydraulic head, L

ρ - density of water, FT^2/L^4

APPENDIX B

SOIL PROFILE LOGS

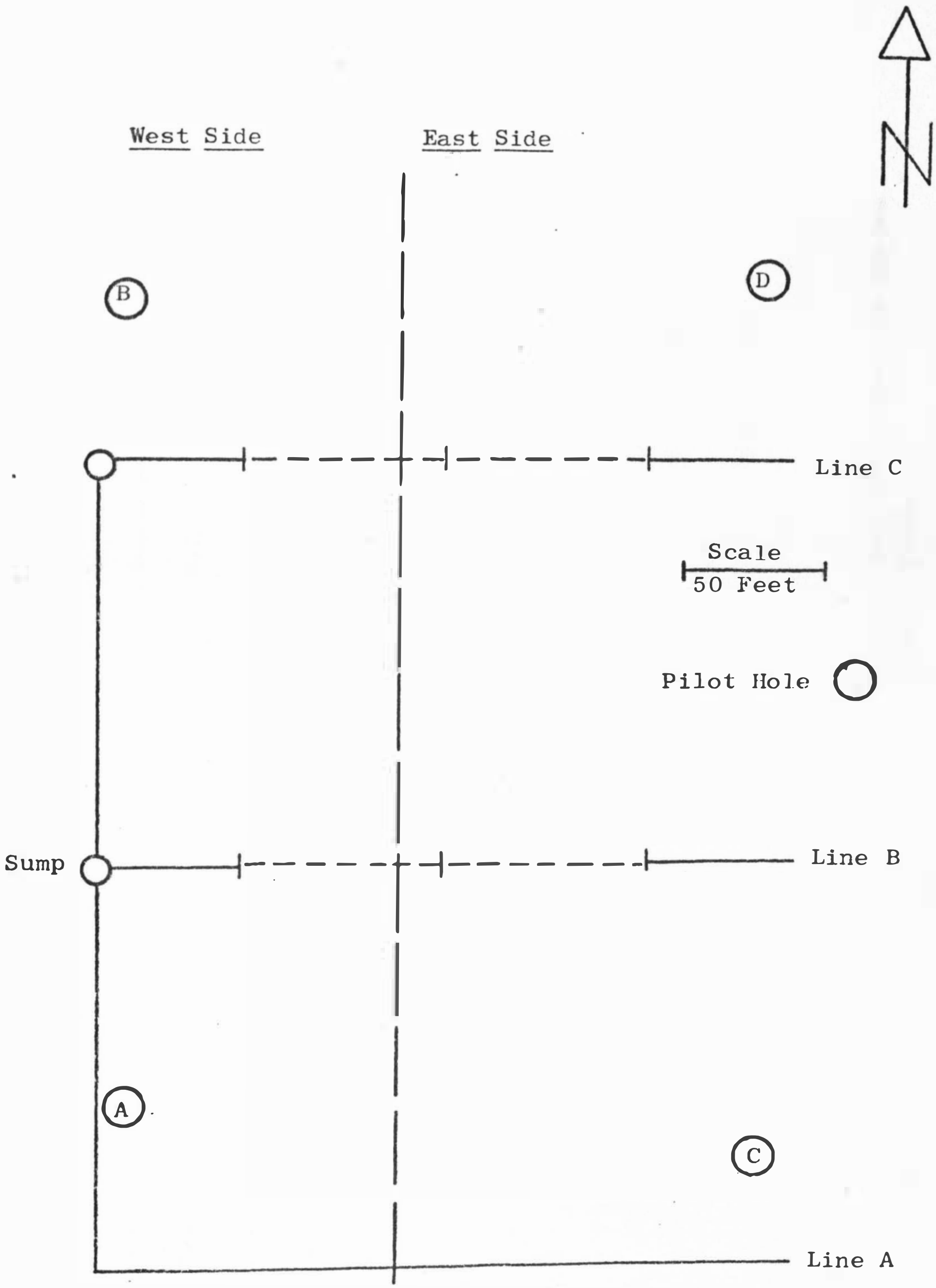

Figure 13. Locations of Pilot Holes for the Soil Profile Log

Table B-1. Soil Profile Log for Site A

Depth (ft)		1	2	3	4	5	6	7	8	9
Estimated Texture		SiL		SiL		SiCL			SiL	
Estimated Percent of Sand		2		2	2			10		
Estimated Percent of Clay		26		24	28		33	12		
Estimated Hydraulic Conductivity (ft/day)		0.8		0.8	0.4		0.2	0.70		
Color		Black		Olive				Lt.Gr.	Ol.Yel.	
Consistency		Friable		Friable	Friable		Firm	Friable		
Structure	Class	Med Fine		Fine	No Structure		Coarse	No Structure		
	Type	Sbk Prismatic		SbK			AbK Platy			
	Grade	Mod		Mod	Mod		Strong	Weak		
Mottling							Few	Few		
Wetness		Moist		FC						
Remarks					Slightly laminated tends to platy		Heavy Fe Stains			

Table B-1. (continued)

Depth (ft)	10	11	12	13	14	15	16	17	18
Estimated Texture				SiL				Clay Till	
Estimated Percent of Sand		15		8					
Estimated Percent of Clay		8		20					
Estimated Hydraulic Conductivity (ft/day)		1.0		0.4		0.2	0.14	0.06	0.04
Color				Olv.Yel			Yel.Br.	Yel.Br.	
Consistency							Firm		
Structure Class				Dense					
Structure Type								Strong	Massive
Structure Grade				Strong					
Mottling				Few					
Wetness				Saturated					
Remarks									

Table B-2. Soil Profile Log for Site B

Depth (ft)		1	2	3	4	5	6	7	8	9
Estimated Texture		SiL		SiCL				SiL	SiL	
Estimated Percent of Sand		5	3					15		
Estimated Percent of Clay		25	27			30	32	8		
Estimated Hydraulic Conductivity (ft/day)		1.0		0.6		0.4			1.0	
Color		Black		Dr.Gr.					Olv.Yel.	
Consistency		Friable		Friable					Friable	
Structure	Class	Med Fine		Med Fine					No structure	
	Type	Crumb Prismatic		Platy						
	Grade	Mod		Mod-strong		Strong			Weak	
Mottling				Few					Few	
Wetness		Moist		FC		Saturated			Saturated	
Remarks				Poor Irregular structure		Fe Stains Slightly Laminated				

Table B-2. (continued)

Depth (ft)		10	11	12	13	14	15	16	17	18
Estimated Texture					SiL				Cl Till	
Estimated Percent of Sand					5					
Estimated Percent of Clay					20		27			
Estimated Hydraulic Conductivity (ft/day)			0.8	0.6	0.4		0.2		0.06	
Color									Yel.Br.	
Consistency										
Structure	Class									
	Type									
	Grade		Moderate			Few thin sandy lenses	Mod-strong		Massive	
Mottling										
Wetness										
Remarks					Fe Stains Dense					

Table B-3. Soil Profile Log for Site C

Depth (ft)		1	2	3	4	5	6	7	8	9
Estimated Texture		SiL	SiCL						SiL	
Estimated Percent of Sand		10	4	2			2		12	
Estimated Percent of Clay		26	27		30		34		8	
Estimated Hydraulic Conductivity (ft/day)		1.0	0.6	0.4		0.50	0.6		0.9	
Color		Black	Olv.Gr.						Yel.Br.	
Consistency		Friable	Friable						Friable	
Structure	Class	Fine	Fine	Med Fine		Coarse			No Structure	
	Type	SbK	AbK	Platy		poor				
	Grade	Mod	Mod		Strong	irregular structure			Weak	
Mottling				Few					Few	
Wetness		Moist	FC		Saturated				Saturated	
Remarks			Flaky	Small pores		Pronounced vertical cleavage	W/Gr. Mottling Few Fe Stains			

Table B-3. (continued)

Depth (ft)		10	11	12	13	14	15	16	17	18
Estimated Texture								SiCL		CL Till
Estimated Percent of Sand		25	20		10	5	5			
Estimated Percent of Clay					18	24	27			
Estimated Hydraulic Conductivity (ft/day)		1.2	1.0	0.6		0.4		0.2	0.16	0.04
Color		Yel.Br.								Yel.Br.
Consistency										
Structure	Class									
	Type									
	Grade			Moderate			Strong	Few thin sand and clay lenses		
Mottling										
Wetness										
Remarks							Few Fe Stains			

Table B-4. Soil Profile Log for Site D

Depth (ft)		1	2	3	4	5	6	7	8	9
Estimated Texture		SiL	SiCL				SiL			
Estimated Percent of Sand		5	4	2			10	12		
Estimated Percent of Clay		26	27	30	32	34	10			
Estimated Hydraulic Conductivity (ft/day)		1.0	0.8	0.4	0.4	0.6	0.8	0.9		
Color		Black	Olv.Gr.			Olv.Gr.	Yel.Br.			
Consistency		Friable	Friable			Firm	Friable			
Structure	Class	Med	Fine			No Str.	No Structure			
	Type	Crumb	AbK	Platy		Dense				
	Grade	Mod	Mod			Strong	Weak			
Mottling					Few		Few			
Wetness		Moist	Moist	FC		Saturated				
Remarks			small pores			pronounced vertical cleavage				

Table B-4. (continued)

Depth (ft)		10	11	12	13	14	15	16	17	18
Estimated Texture								SiCL		CL Till
Estimated Percent of Sand		12		10		17	8	2		
Estimated Percent of Clay		15		18		12	20	28	30	
Estimated Hydraulic Conductivity (ft/day)		0.8		0.7		1.0	0.5	0.04		0.02
Color								Yel.Br.		Yel.Br.
Consistency								Firm		Firm
Structure	Class									
	Type									
	Grade			Mod.		coarser lense		Strong		Strong
Mottling										
Wetness										
Remarks										

APPENDIX C

HYDRAULIC CONDUCTIVITY DATA

Table C-1. Auger Hole Hydraulic Conductivity for the Undisturbed Soil at the Three to Seven Foot Depth

East Side		West Side	
Soil Texture a)	Hydraulic Conductivity (feet/day)	Soil Texture a)	Hydraulic Conductivity (feet/day)
Silty Clay Loam	1.42	Silt Loam	2.76
Silty Clay Loam	5.12	Silty Clay Loam	0.41
Silty Clay Loam	2.44	Silty Clay Loam	3.06
	1.07	Silt Loam	0.33
	1.92	Silty Clay Loam	1.23
Silt Loam	1.92	Silty Clay Loam	1.73
Silt Loam	3.04	Silt Loam	3.20
	0.55	Silty Clay Loam	1.35
	1.00	Silt Loam	1.95
Average	2.06	Average	1.88

a) Estimated by Bureau of Reclamation personnel during field tests

Table C-2. Auger Hole Hydraulic Conductivity for the Backfill Soil at the Three to Seven Foot Depth

East Side	West Side
Hydraulic Conductivity (feet/day)	Hydraulic Conductivity (feet/day)
0.09	0.25
1.98	0.07
0.56	0.85
4.82	0.04
0.50	0.46
0.61	1.18
Average 1.43	Average 0.47

Table C-3. Auger Hole Hydraulic Conductivity for the West Side Undisturbed Soil at the Seven to Ten Foot Depth

	Hydraulic Conductivity (feet/day)
	6.88
	4.66
	2.28
	1.00
	1.95
Average	3.35

Table C-4. Pump-in Hydraulic Conductivity for the Undisturbed Soil at the Three to Seven Foot Depth

East Side		West Side	
Texture a)	Hydraulic Conductivity (feet/day)	Texture a)	Hydraulic Conductivity (feet/day)
Silty Clay Loam	0.84	Silt Loam	0.20
Silt Loam	2.56	Silt Loam	0.48
Silty Clay Loam	0.29	Silty Clay Loam	0.50
Silt Loam	0.36	Silty Clay Loam	0.56
Silt Loam	2.26	Silty Clay Loam	0.70
Silty Clay Loam	0.52	Silty Clay Loam	0.74
Silt Loam	1.22	Silty Clay Loam	0.88
Silt Loam	1.50	Silty Clay Loam	1.02
Silt Loam	1.06	Silt Loam	1.16
Average	1.18	Average	0.70

a) Estimated by Bureau of Reclamation personnel during field tests

Table C-5. Pump-in Hydraulic Conductivity for the Undisturbed Soil at the Seven to Ten Foot Depth

East Side		West Side	
Texture a)	Hydraulic Conductivity (feet/day)	Texture a)	Hydraulic Conductivity (feet/day)
Silt Loam	2.26	Silt Loam	1.96
Silt Loam	1.76	Silt Loam	1.50
Silt Loam	1.42	Silt Loam	1.42
Silt Loam	1.72	Silt Loam	4.32
Silt Loam	1.64	Silt Loam	1.58
Silt Loam	1.70	Silt Loam	1.30
Silt Loam	2.00	Silt Loam	1.38
Silt Loam	1.44	Silt Loam	1.62
Silt Loam	1.84	Silt Loam	1.46
Average	1.91	Average	1.68

a) Estimated by Bureau of Reclamation personnel during field tests

Table C-6. Drain Line Hydraulic Conductivity at the Three to Ten Foot Depth

Date	East Side		West Side	
	Drainable Porosity (ft^3/ft^3)	Hydraulic Conductivity (feet/day)	Drainable Porosity (ft^3/ft^3)	Hydraulic Conductivity (feet/day)
July, 1968	0.030	0.89	0.037	0.86
August, 1968	0.030	0.94	0.033	1.26
August, 1968	0.029	0.85	0.045	0.84
August, 1970	0.018	0.84	0.025	1.20
September, 1971	0.035	0.93	0.041	1.09
October, 1972	0.026	0.79	0.035	1.21
August, 1973	0.016	0.77	0.032	1.10
Average	0.026	0.86	0.035	1.08

APPENDIX D

SOIL WATER HYDRAULIC HEADS

Table D-1. Hydraulic Heads (feet of water) for the SW Piezometer Bank

Depth (ft)	Distance (ft)	Time (min)	Initial Head	Time (min)	Head (ft)	Time (min)	Head (ft)	Time (min)	Head (ft)	Time (min)	Head (ft)	Time (min)	Head (ft)	Time (min)	Head (ft)	Time (min)	Head (ft)
6	2	0	3.65	2	3.15	20	1.95	48	1.75								
6	2	0	3.87	5	3.67	24	3.77	52	2.97								
6	2	0	3.65	7	2.35	29	2.05										
6	4	0	3.60	2	2.25												
6	4	0	3.65	5	2.55	24	1.95	52	1.85	117	1.65	165	1.65				
6	4	0	3.68	8	2.53	29	2.03	55	2.03	119	1.83						
7	2	0	3.53														
7	2	0	3.60	5	2.45	25	1.80	52	1.70	117	1.20	165	1.40	436	1.25		
7	2	0	3.64	8	1.14	29	1.04	56	0.94	120	1.04						
7	4	0	3.60	3	2.35	21	1.75	48	1.55	114	1.40	162	1.35	433	1.20	720	1.05
7	4	0	3.72	6	2.72	25	2.22	53	2.22	118	1.82	166	1.72	437	1.57	723	1.42
7	4	0	3.63	8	2.38	30	2.08	56	1.98	120	1.78	168	1.68	440	1.53	728	1.38
8	2	0	3.46														
8	2	0	3.64	6	3.39	26	3.29	53	3.39	118	3.39	166	3.19	437	3.09	724	2.89
8	2	0	3.82	9	3.57	30	3.57	57	3.47	120	3.47	168	3.47	441	3.17	729	2.87
8	4	0	3.53	4	2.63	22	1.83	49	1.53	115	1.53	163	1.43	432	1.28	719	1.23
8	4	0	3.73	6	2.53	26	2.28	53	2.08	118	1.88	166	1.88	437	1.68	724	1.48
8	4	0	3.75	9	2.45	31	2.20	57	2.10	121	1.70	168	1.95	441	1.70	729	1.60
9	2	0	3.31	4	3.21	22	3.01	50	2.91	115	2.81	163	2.71	433	2.21	720	1.71
9	2	0	3.60	6	1.50	26	1.30	53	1.35	118	1.05	166	1.05	438	0.95	725	0.85
9	2	0	3.87	9	3.57	31	3.37	57	3.57	121	3.37	168	3.27	442	2.77	730	2.47
9	4	0	3.57	4	3.62	23	3.42	51	3.42	116	3.42	164	3.42	433	2.97	720	2.57
9	4	0	3.67	7	2.57	27	2.52	54	1.92	118	2.02	167	1.62	438	1.57	725	1.52
9	4	0	4.24	9	4.14	31	4.09	58	4.09	121	3.69	168	3.59	442	3.89	730	3.69

Table D-1. (continued)

Depth (ft)	Distance (ft)	Time (min)	Initial Head	Time (min)	Head (ft)	Time (min)	Head (ft)	Time (min)	Head (ft)	Time (min)	Head (ft)	Time (min)	Head (ft)	Time (min)	Head (ft)	Time (min)	Head (ft)
10	2	0	3.66	4	1.66	23	1.26	51	1.26	116	1.26	164	1.26	434	1.16	721	1.01
10	2	0	3.86	7	3.61	28	3.61	54	3.61	119	3.61	167	3.51	338	3.16	726	2.71
10	2	0	3.77	10	2.32	32	2.12	58	2.02	122	1.92	169	1.92	443	1.72	731	1.62
10	4	0	3.74	5	3.49	24	3.24	52	3.04	117	2.54	165	2.44	435	1.94	721	1.74
10	4	0	3.73	7	2.68	28	2.58	55	2.38	119	2.18	167	2.08	439	1.88	726	1.73
10	4	0	3.80	10	2.65	32	2.50	58	2.50	122	2.20	169	2.00	443	1.90	732	1.80

Table D-2. Hydraulic Heads (feet of Water) for the NW Piezometer Bank

Depth (ft)	Distance (ft)	Time (min)	Initial Head	Time (min)	Head (ft)	Time (min)	Head (ft)	Time (min)	Head (ft)	Time (min)	Head (ft)	Time (min)	Head (ft)	Time (min)	Head (ft)	Time (min)	Head (ft)
6	2	0	3.13	2	2.63	38	1.68										
6	2	0	3.24	11	3.24	44	2.34	83	2.29	135	2.04	181	1.94	467	1.94		
6	2	0	3.19	16	3.09	49	2.94	88	2.69	137	2.59						
6	4	0	3.19	3	2.89	39	2.19	78	1.89								
6	4	0	3.39	11	3.14	44	2.79	83	2.09								
6	4	0	3.31	16	2.81	50	2.26	88	2.11	137	2.01	183	2.01	472	1.76		
7	2	0	3.19	5	2.69	40	1.94	79	1.74	133	1.49	179	1.39	462	1.39	750	1.39
7	2	0	3.21	12	2.56	45	2.11	84	1.96	135	1.86	181	1.81	468	1.56	755	1.51
7	2	0	3.20	17	2.60	51	2.05	89	2.00	137	1.60	183	1.85	473	1.60	760	1.60
7	4	0	3.26	7	2.96	41	2.31	80	2.16	133	1.96	179	1.96	462	1.61	750	1.71
7	4	0	3.35	12	2.90	45	2.40	85	2.30	135	2.15	181	2.05	468	1.85	755	1.80
7	4	0	3.40	17	2.95	52	2.55	89	2.40	137	2.30	183	2.25	474	1.90	760	1.85
8	2	0	3.62	8	3.72	41	3.67	80	3.72	133	3.42	180	3.52	463	3.32	751	3.32
8	2	0	3.33	13	2.73	45	2.23	85	2.08	136	2.03	182	1.93	469	1.73	756	1.73
8	2	0	3.09	18	3.29	52	3.24	90	3.29	138	3.29	184	3.19	474	2.89	761	2.54
8	4	0	3.36	8	3.16	42	2.41	81	2.41	134	2.16	180	2.06	463	1.91	751	1.86
8	4	0	3.39	13	3.09	46	2.64	86	2.39	136	2.29	182	2.24	469	2.04	756	1.89
8	4	0	3.3[illegible]	18	2.90	53	2.45	90	2.40	138	2.20	184	2.30	475	1.90	761	1.90
9	2	0	3.63	9	3.73	42	3.43	81	3.53	134	3.33	180	3.23	464	3.23	752	3.13
9	2	0	3.31	14	2.61	46	2.11	86	2.16	136	2.11	182	2.01	469	1.81	757	1.71
9	2	0	3.34	19	3.29	53	3.14	91	2.94	138	2.69	184	2.69	475	2.19	762	1.89
9	4	0	3.44	9	3.39	43	2.59	81	3.14	134	2.34	180	2.24	465	1.94	752	1.84
9	4	0	3.41	15	2.86	46	2.46	87	2.31	136	2.21	182	2.16	470	1.91	757	1.81
9	4	0	3.35	19	3.25	54	3.15	91	2.90	139	2.65	184	2.55	476	2.05	762	1.85

Table D-2. (continued)

Depth (ft)	Distance (ft)	Time (min)	Initial Head	Time (min)	Head (ft)	Time (min)	Head (ft)	Time (min)	Head (ft)	Time (min)	Head (ft)	Time (min)	Head (ft)	Time (min)	Head (ft)	Time (min)	Head (ft)
10	2	0	3.38	10	3.33	43	3.18	82	3.38	134	2.78	180	2.68	466	2.28	753	1.93
10	2	0	3.44	15	3.49	47	2.14	87	2.04	136	1.99	182	1.99	470	1.69	758	1.49
10	2	0	3.26	20	1.96	54	1.81	92	1.71	139	1.56	185	1.56	477	1.36	763	1.36
10	4	0	3.26	10	3.46	44	3.31	82	1.96	135	3.16	181	3.16	467	2.86	753	2.61
10	4	0	3.69	15	3.89	48	3.84	87	3.84	137	3.79	183	3.84	471	3.69	758	3.69
10	4	0	3.26	20	2.56	55	2.36	93	2.31	140	2.26	185	2.16	478	1.86	763	1.86

Table D-3. Hydraulic Heads (feet of water) for the NE Piezometer Bank

Depth (ft)	Distance (ft)	Time (min)	Initial Head	Time (min)	Head (ft)	Time (min)	Head (ft)	Time (min)	Head (ft)	Time (min)	Head (ft)	Time (min)	Head (ft)	Time (min)	Head (ft)	Time (min)	Head (ft)
6	2	0	2.70	22	1.90	56	1.75	93	1.60								
6	2	0	2.70	27	1.95	61	1.80										
6	2	0	2.75	31	2.15	68	2.00	104	2.05								
6	4	0	2.67	22	2.02	57	1.82	93	1.72	140	1.57						
6	4	0	2.76	27	2.46	62	2.41	98	2.26								
6	4	0	2.72	32	2.32	68	2.22	104	2.12								
7	2	0	2.75	23	1.70	57	1.55	94	1.45	141	1.25	189	1.25	480	0.90	764	0.95
7	2	0	2.77	28	1.87	62	1.62	99	1.57	143	1.52	192	1.22	483	1.22	768	1.12
7	2	0	2.95	32	2.80	69	2.85	104	2.85	146	2.75	195	2.55	487	2.55	771	2.45
7	4	0	2.83	23	2.13	58	1.93	95	1.78	141	1.68	189	1.58	480	1.28	765	1.23
7	4	0	2.79	28	2.19	63	1.99	100	1.84	143	1.74	192	1.64	484	1.44	768	1.34
7	4	0	2.68	33	2.53	69	2.53	104	2.38	146	2.33	195	2.28	487	1.68	771	1.38
8	2	0	2.88	24	2.83	58	2.78	96	2.73	142	2.68	190	2.68	480	2.38	765	2.18
8	2	0	2.97	28	2.97	64	2.87	101	2.82	144	2.82	193	2.62	484	2.62	768	2.52
8	2	0	2.66	33	2.36	70	2.06	104	1.86	147	1.76	196	1.51	488	1.21	772	1.06
8	4	0	2.82	24	2.17	59	1.92	96	1.82	142	1.72	190	1.72	481	1.42	765	1.32
8	4	0	2.80	29	2.25	65	2.00	101	1.90	144	1.80	193	1.80	484	1.50	769	1.30
8	4	0	2.88	34	2.88	70	2.78	105	2.70	147	2.68	196	2.53	488	2.48	772	2.48
9	2	0	2.83	25	1.78	59	1.53	96	1.53	142	1.53	191	1.43	481	1.23	766	1.13
9	2	0	2.97	29	2.92	66	2.77	102	2.77	144	2.82	193	2.77	485	2.47	769	2.37
9	2	0	2.93	34	3.03	71	2.93	105	2.93	147	2.83	196	2.83	488	2.93	772	2.83
9	4	0	2.81	25	2.71	60	2.61	97	2.46	142	2.41	191	2.21	481	1.71	766	1.51
9	4	0	2.86	30	2.21	66	2.01	102	1.96	145	1.86	194	1.76	485	1.66	769	1.46
9	4	0	2.70	35	2.10	71	1.95	105	1.85	147	1.85	196	1.75	489	1.45	773	1.35

Table D-3. (continued)

Depth (ft)	Distance (ft)	Time (min)	Initial Head	Time (min)	Head (ft)	Time (min)	Head (ft)	Time (min)	Head (ft)	Time (min)	Head (ft)	Time (min)	Head (ft)	Time (min)	Head (ft)	Time (min)	Head (ft)
10	2	0	2.95	26	2.85	60	2.75	97	2.65	142	2.55	191	2.55	482	2.15	766	1.85
10	2	0	2.83	30	2.83	67	2.73	103	2.73	145	2.63	194	2.53	485	2.23	770	1.98
10	2	0	2.74	35	2.59	72	2.44	105	2.44	148	2.44	196	2.34	489	1.74	773	1.54
10	4	0	2.69	26	2.24	61	2.04	97	1.99	143	1.89	192	1.79	482	1.59	767	1.44
10	4	0	2.83	31	2.68	67	2.58	103	2.53	146	2.48	195	2.38	486	1.93	770	1.68
10	4	0	3.06	36	2.91	72	2.91	106	2.91	148	2.91	197	2.86	490	2.66	774	2.56

Table D-4. Hydraulic Heads (feet of water) for the SE Piezometer Bank

Depth (ft)	Distance (ft)	Time (min)	Initial Head	Time (min)	Head (ft)	Time (min)	Head (ft)	Time (min)	Head (ft)	Time (min)	Head (ft)	Time (min)	Head (ft)	Time (min)	Head (ft)	Time (min)	Head (ft)
6	2	0	3.36	11	2.11	33	2.06	59	2.06	123	2.06						
6	2	0	3.49	13	2.79	37	2.49	63	2.29								
6	2	0	3.39	15	2.79	40	2.59										
6	4	0	3.47	11	2.77	33	2.57	59	2.27	123	2.17						
6	4	0	3.29	13	2.79	37	2.59	63	2.29			162	2.19				
6	4	0	3.31	15	2.91	40	2.81	66	2.51								
7	2	0	3.44	11	2.64	33	2.54	59	2.34	123	2.14	160	1.94	445	1.84	734	1.64
7	2	0	3.47	13	2.62	37	2.42	64	2.32	125	2.02	162	1.92	449	1.72	739	1.52
7	2	0	3.23	16	2.63	40	2.23	66	2.13	128	2.03	165	2.03			744	1.63
7	4	0	3.41	11	2.71	34	2.61	59	2.41	123	2.21	160	2.11	445	1.81	734	1.71
7	4	0	3.41	13	2.71	37	2.56	64	2.46	126	1.76	163	2.16	450	1.86	739	1.71
7	4	0	3.29	16	2.69	40	2.54	66	2.44	128	2.04	165	2.24	454	1.84	744	1.74
8	2	0	3.47	12	2.52	34	2.32	60	2.07	123	1.92	160	1.92	446	1.72	735	1.62
8	2	0	3.40	14	2.35	38	2.15	65	2.15	126	1.95	163	1.95	450	1.65	740	1.55
8	2	0	3.48	16	3.23	41	3.23	67	3.23	128	3.23	165	3.13	454	2.53	744	2.33
8	4	0	3.50	12	2.75	35	2.50	60	2.50	124	2.20	161	2.10	446	1.90	735	1.80
8	4	0	3.43	14	2.88	38	2.78	65	2.58	127	2.48	164	2.48	450	1.98	740	1.93
8	4	0	3.40	17	2.95	41	2.65	67	2.45	129	2.35	166	2.35	455	1.95	745	1.85
9	2	0	3.43	12	2.53					124	1.93	161	1.83	446	1.43	736	1.33
9	2	0	3.75	14	3.70	38	3.70	65	3.70	127	3.70	164	3.70	451	3.70	741	3.70
9	2	0	4.04	17	3.54	42	3.74	68	3.74	129	3.59	166	3.49	455	3.74	745	3.64
9	4	0	3.89	12	3.69	36	3.69	61	3.69	124	3.59	161	3.39	447	3.49	736	3.69
9	4	0	3.42	14	2.77	39	2.72	65	2.32	127	2.32	164	2.22	451	2.02	741	1.92
9	4	0	4.29	18	4.09	42	4.29	68	4.29	129	4.09	166	4.29	456	4.14	745	4.19

Table D-4. (continued)

Depth (ft)	Distance (ft)	Time (min)	Initial Head	Time (min)	Head (ft)	Time (min)	Head (ft)	Time (min)	Head (ft)	Time (min)	Head (ft)	Time (min)	Head (ft)	Time (min)	Head (ft)	Time (min)	Head (ft)
10	2	0	3.60	13	2.90	36	2.80	62	2.60	124	2.50	161	2.40	447	2.20	737	1.95
10	2	0	3.52	14	3.37	39	3.27	65	3.07	127	2.87	164	2.87	451	2.37	742	2.17
10	2	0	4.34	18	3.89	43	4.39	69	4.39	130	4.19	166	4.29	456	4.24	746	4.24
10	4	0	3.52	13	2.92	36	2.82	63	2.72	124	2.32	161	2.42	448	2.22	737	1.92
10	4	0	3.47	15	3.37	39	3.37	66	3.17	127	2.97	164	2.92	452	2.47	742	2.17
10	4	0	4.36	19	3.81	43	3.81	69	4.31	130	4.31	167	4.31	457	4.31	746	4.26

APPENDIX E

SOIL WATER HYDRAULIC HEAD RATIOS

Table E-1. Hydraulic Head Ratios for the Piezometric Data of the SW Piezometer Bank

Depth (ft)	Distance (ft)	Time (min)	Ratio	Time (min)	Ratio	Time (min)	Ratio	Time (min)	Ratio	Time (min)	Ratio	Time (min)	Ratio	Time (min)	Ratio	Time (min)	Ratio
6 a)	2	0	1.00	2	0.86	20	0.53	48	0.48								
6 a)	2	0	1.00	5	0.94	24	0.97	52	0.77								
6 a)	2	0	1.00	7	0.64	29	0.56										
6 a)	4	0	1.00	2	0.62												
6 a)	4	0	1.00	5	0.70	24	0.53	52	0.51	117	0.45	165	0.45				
6 a)	4	0	1.00	8	0.69	29	0.55	55	0.55	119	0.50						
7 a)	2	0	1.00														
7	2	0	1.00	5	0.68	25	0.50	52	0.47	117	0.33	165	0.39	436	0.35		
7	2	0	1.00	8	0.31	29	0.29	56	0.26	120	0.29						
7	4	0	1.00	3	0.65	21	0.49	48	0.43	114	0.39	162	0.40	433	0.33	720	0.29
7	4	0	1.00	6	0.73	25	0.60	53	0.60	118	0.49	166	0.46	437	0.42	723	0.38
7	4	0	1.00	8	0.66	30	0.57	56	0.55	120	0.49	168	0.46	440	0.42	728	0.38
8 a)	2	0	1.00														
8	2	0	1.00	6	0.93	26	0.90	53	0.93	118	0.93	166	0.88	437	0.85	724	0.79
8	2	0	1.00	9	0.93	30	0.93	57	0.91	120	0.91	168	0.91	441	0.83	729	0.75
8	4	0	1.00	4	0.75	22	0.52	49	0.43	115	0.43	163	0.41	432	0.36	719	0.35
8	4	0	1.00	6	0.68	26	0.61	53	0.56	118	0.50	166	0.50	437	0.45	724	0.40
8	4	0	1.00	9	0.65	31	0.59	57	0.56	121	0.45	168	0.52	441	0.45	729	0.43
9	2	0	1.00	4	0.97	22	0.91	50	0.88	115	0.85	163	0.82	433	0.67	720	0.52
9	2	0	1.00	6	0.42	26	0.36	53	0.37	118	0.29	166	0.29	438	0.26	725	0.24
9	2	0	1.00	9	0.92	31	0.87	57	0.92	121	0.87	168	0.84	442	0.72	730	0.64
9	4	0	1.00	4	1.01	23	0.96	51	0.96	116	0.96	164	0.96	433	0.83	720	0.72
9	4	0	1.00	7	0.70	27	0.69	54	0.52	118	0.55	167	0.44	438	0.43	725	0.41
9	4	0	1.00	9	0.98	31	0.96	58	0.96	121	0.87	168	0.85	442	0.92	730	0.87

Table E-1. (continued)

Depth (ft)	Distance (ft)	Time (min)	Ratio	Time (min)	Ratio	Time (min)	Ratio	Time (min)	Ratio	Time (min)	Ratio	Time (min)	Ratio	Time (min)	Ratio	Time (min)	Ratio
10	2	0	1.00	4	0.45	23	0.34	51	0.34	116	0.34	164	0.34	434	0.32	721	0.28
10	2	0	1.00	7	0.94	28	0.94	54	0.94	119	0.94	167	0.91	438	0.82	726	0.70
10	2	0	1.00	10	0.62	32	0.56	58	0.54	122	0.51	169	0.51	443	0.46	731	0.43
10	4	0	1.00	5	0.93	24	0.87	52	0.81	117	0.68	165	0.65	435	0.52	721	0.47
10	4	0	1.00	7	0.72	28	0.69	55	0.64	119	0.58	167	0.56	439	0.50	726	0.46
10	4	0	1.00	10	0.70	32	0.66	58	0.66	122	0.58	169	0.53	443	0.50	732	0.47

a) Set of data not used in statistical analyses

Table E-2. Hydraulic Head Ratios for the Piezometric Data of the NW Piezometer Bank

Depth (ft)	Distance (ft)	Time (min)	Ratio	Time (min)	Ratio	Time (min)	Ratio	Time (min)	Ratio	Time (min)	Ratio	Time (min)	Ratio	Time (min)	Ratio	Time (min)	Ratio
6 a)	2	0	1.00	2	0.84	38	0.54										
6 a)	2	0	1.00	11	1.00	44	0.72	83	0.71	135	0.63	181	0.60	467	0.60		
6 a)	2	0	1.00	16	0.97	49	0.92	88	0.84	137	0.81						
6 a)	4	0	1.00	3	0.91	39	0.69	78	0.59								
6 a)	4	0	1.00	11	0.93	44	0.82	83	0.62								
6 a)	4	0	1.00	16	0.85	50	0.68	88	0.64	137	0.61	183	0.61	472	0.53		
7	2	0	1.00	5	0.84	40	0.61	79	0.55	133	0.47	179	0.44	462	0.44	750	0.44
7	2	0	1.00	12	0.80	45	0.66	84	0.61	135	0.58	181	0.56	468	0.49	755	0.47
7	2	0	1.00	17	0.81	51	0.64	89	0.62	137	0.50	183	0.58	473	0.50	760	0.50
7	4	0	1.00	7	0.91	41	0.71	80	0.66	133	0.60	179	0.60	462	0.49	750	0.52
7	4	0	1.00	12	0.87	45	0.72	85	0.69	135	0.64	181	0.61	468	0.55	755	0.54
7	4	0	1.00	17	0.87	52	0.75	89	0.71	137	0.68	183	0.66	474	0.56	760	0.54
8	2	0	1.00	8	1.03	41	1.01	80	1.03	133	0.94	180	0.97	463	0.92	751	0.92
8	2	0	1.00	13	0.82	45	0.67	85	0.62	136	0.61	182	0.58	469	0.52	756	0.52
8	2	0	1.00	18	1.06	52	1.05	90	1.06	138	1.06	184	1.03	474	0.94	761	0.82
8	4	0	1.00	8	0.94	42	0.72	81	0.72	134	0.64	180	0.61	463	0.57	751	0.55
8	4	0	1.00	13	0.91	46	0.78	86	0.71	136	0.68	182	0.66	469	0.60	756	0.56
8	4	0	1.00	18	0.88	53	0.74	90	0.73	138	0.67	184	0.70	475	0.58	761	0.58
9	2	0	1.00	9	1.03	42	0.94	81	0.97	134	0.92	180	0.89	464	0.89	752	0.86
9	2	0	1.00	14	0.79	46	0.64	86	0.65	136	0.64	182	0.61	469	0.55	757	0.52
9	2	0	1.00	19	0.99	53	0.94	91	0.88	138	0.81	184	0.81	475	0.66	762	0.57
9	4	0	1.00	9	0.99	43	0.75	81	0.91	134	0.68	180	0.65	465	0.56	752	0.53
9	4	0	1.00	15	0.84	46	0.72	87	0.68	136	0.65	182	0.63	470	0.56	757	0.53
9	4	0	1.00	19	0.97	54	0.94	91	0.87	139	0.79	184	0.76	476	0.61	762	0.55

Table E-2. (continued)

Depth (ft)	Distance (ft)	Time (min)	Ratio	Time (min)	Ratio	Time (min)	Ratio	Time (min)	Ratio	Time (min)	Ratio	Time (min)	Ratio	Time (min)	Ratio	Time (min)	Ratio
	2	0	1.00	10	0.99	43	0.94	82	1.00	134	0.82	180	0.79	466	0.67	753	0.57
10	2	0	1.00	15	1.01	47	0.62	87	0.59	136	0.58	182	0.58	470	0.49	758	0.43
10	2	0	1.00	20	0.60	54	0.56	92	0.52	139	0.48	185	0.48	477	0.42	763	0.42
10	4	0	1.00	10	1.06	44	1.02	82	0.60	135	0.97	181	0.97	467	0.88	753	0.80
10 a)	4	0	1.00	15	1.05	48	1.04	87	1.04	137	1.03	183	1.04	471	1.00	758	1.00
10	4	0	1.00	20	0.79	55	0.72	93	0.71	140	0.69	185	0.66	478	0.57	763	0.57

a) Set of data not used in the statistical analyses

Table E-3. Hydraulic Head Ratios for the Piezometric Data of the NE Piezometer Bank

Depth (ft)	Distance (ft)	Time (min)	Ratio	Time (min)	Ratio	Time (min)	Ratio	Time (min)	Ratio	Time (min)	Ratio	Time (min)	Ratio	Time (min)	Ratio	Time (min)	Ratio
6 a)	2	0	1.00	22	0.70	56	0.65	93	0.59								
6 a)	2	0	1.00	27	0.72	61	0.67										
6 a)	2	0	1.00	31	0.78	68	0.73	104	0.75								
6 a)	4	0	1.00	22	0.76	57	0.68	93	0.64	140	0.59						
6 a)	4	0	1.00	27	0.89	62	0.87	98	0.82								
6 a)	4	0	1.00	32	0.85	68	0.82	104	0.78								
7	2	0	1.00	23	0.62	57	0.56	94	0.53	141	0.45	189	0.45	480	0.33	764	0.35
7	2	0	1.00	28	0.68	62	0.58	99	0.57	143	0.55	192	0.44	483	0.44	768	0.40
7	2	0	1.00	32	0.95	69	0.97	104	0.97	146	0.93	195	0.86	487	0.86	771	0.83
7	4	0	1.00	23	0.75	58	0.68	95	0.63	141	0.59	189	0.56	480	0.45	765	0.43
7	4	0	1.00	28	0.78	63	0.71	100	0.66	143	0.62	192	0.59	484	0.52	768	0.48
7	4	0	1.00	33	0.94	69	0.94	104	0.89	146	0.87	195	0.85	487	0.63	771	0.51
8	2	0	1.00	24	0.98	58	0.97	96	0.95	142	0.93	190	0.93	480	0.83	765	0.76
8	2	0	1.00	28	1.00	64	0.97	101	0.95	144	0.95	193	0.88	484	0.88	768	0.85
8	2	0	1.00	33	0.89	70	0.77	104	0.70	147	0.66	196	0.57	488	0.45	772	0.40
8	4	0	1.00	24	0.77	59	0.68	96	0.65	142	0.61	190	0.61	481	0.50	765	0.47
8	4	0	1.00	29	0.80	65	0.71	101	0.68	144	0.64	193	0.64	484	0.54	769	0.46
8	4	0	1.00	34	1.00	70	0.97	105	0.97	147	0.93	196	0.88	488	0.86	772	0.86
9	2	0	1.00	25	0.63	59	0.54	96	0.54	142	0.54	191	0.51	481	0.43	766	0.40
9	2	0	1.00	29	0.98	66	0.93	102	0.93	144	0.95	193	0.93	485	0.83	769	0.80
9 a)	2	0	1.00	34	1.03	71	1.00	105	1.00	147	0.97	196	0.97	488	1.00	772	0.97
9	4	0	1.00	25	0.96	60	0.93	97	0.88	142	0.86	191	0.79	481	0.61	766	0.54
9	4	0	1.00	30	0.77	66	0.70	102	0.69	145	0.65	194	0.62	485	0.58	769	0.51
9	4	0	1.00	35	0.78	71	0.72	105	0.69	147	0.69	196	0.65	489	0.54	773	0.50

Table E-3. (continued)

Depth (ft)	Distance (ft)	Time (min)	Ratio	Time (min)	Ratio	Time (min)	Ratio	Time (min)	Ratio	Time (min)	Ratio	Time (min)	Ratio	Time (min)	Ratio	Time (min)	Ratio
10	2	0	1.00	26	0.97	60	0.93	97	0.90	142	0.86	191	0.86	482	0.73	766	0.63
10	2	0	1.00	30	1.00	67	0.96	103	0.96	145	0.93	194	0.89	485	0.79	770	0.70
10	2	0	1.00	35	0.95	72	0.89	105	0.89	148	0.89	196	0.85	489	0.64	773	0.56
10	4	0	1.00	26	0.83	61	0.76	97	0.74	143	0.70	192	0.67	482	0.59	767	0.54
10	4	0	1.00	31	0.95	67	0.91	103	0.89	146	0.88	195	0.84	486	0.68	770	0.59
10	4	0	1.00	36	0.95	72	0.95	106	0.95	148	0.95	197	0.93	490	0.87	774	0.84

a) Set of data not used in the statistical analysis

Table E-4. Hydraulic Head Ratios for the Piezometric Data of the SE Piezometer Bank

Depth (ft)	Distance (ft)	Time (min)	Ratio	Time (min)	Ratio	Time (min)	Ratio	Time (min)	Ratio	Time (min)	Ratio	Time (min)	Ratio	Time (min)	Ratio	Time (min)	Ratio
6 a)	2	0	1.00	11	0.63	33	0.61	59	0.64	123	0.61						
6 a)	2	0	1.00	13	0.80	37	0.71	63	0.66								
6 a)	2	0	1.00	15	0.82	40	0.76										
6 a)	4	0	1.00	11	0.80	33	0.74	59	0.65	123	0.63						
6 a)	4	0	1.00	13	0.85	37	0.79	63	0.70			162	0.67				
6 a)	4	0	1.00	15	0.88	40	0.85	66	0.76								
7	2	0	1.00	11	0.77	33	0.74	59	0.68	123	0.62	160	0.56	445	0.53	734	0.48
7	2	0	1.00	13	0.76	37	0.70	64	0.67	125	0.58	162	0.55	449	0.50	739	0.44
7	2	0	1.00	16	0.81	40	0.69	66	0.66	128	0.63	165	0.63			744	0.50
7	4	0	1.00	11	0.79	34	0.77	59	0.71	123	0.65	160	0.62	445	0.53	734	0.50
7	4	0	1.00	13	0.79	37	0.75	64	0.72	126	0.52	163	0.63	450	0.55	739	0.50
7	4	0	1.00	16	0.82	40	0.77	66	0.74	128	0.62	165	0.68	454	0.56	744	0.53
8	2	0	1.00	12	0.73	34	0.67	60	0.60	123	0.55	160	0.55	446	0.50	735	0.47
8	2	0	1.00	14	0.69	38	0.63	65	0.63	126	0.57	163	0.57	450	0.49	740	0.46
8	2	0	1.00	16	0.93	41	0.93	67	0.93	128	0.93	165	0.90	454	0.73	744	0.67
8	4	0	1.00	12	0.79	35	0.71	60	0.71	124	0.63	161	0.60	446	0.54	735	0.51
8	4	0	1.00	14	0.84	38	0.81	65	0.75	127	0.72	164	0.72	450	0.58	740	0.56
8	4	0	1.00	17	0.87	41	0.78	67	0.72	129	0.69	166	0.69	455	0.57	745	0.54
9	2	0	1.00	12	0.74					124	0.56	161	0.53	446	0.42	736	0.39
9	2	0	1.00	14	0.99	38	0.99	65	0.99	127	1.01	164	0.99	451	0.99	741	0.99
9 a)	2	0	1.00	17	0.88	42	0.93	68	0.93	129	0.89	166	0.86	.55	0.93	745	0.90
9	4	0	1.00	12	0.95	36	1.00	61	0.95	124	0.92	161	0.87	447	0.90	736	0.95
9	4	0	1.00	14	0.81	39	0.80	65	0.68	127	0.68	164	0.65	451	0.59	741	0.56
9 a)	4	0	1.00	18	0.95	42	1.00	68	1.00	129	0.95	166	1.00	456	0.97	745	0.98

Table E-4. (continued)

Depth (ft)	Distance (ft)	Time (min)	Ratio	Time (min)	Ratio	Time (min)	Ratio	Time (min)	Ratio	Time (min)	Ratio	Time (min)	Ratio	Time (min)	Ratio	Time (min)	Ratio
10	2	0	1.00	13	0.81	36	0.78	62	0.72	124	0.69	161	0.67	447	0.61	737	0.54
10	2	0	1.00	14	0.96	39	0.93	65	0.87	127	0.82	164	0.82	451	0.67	742	0.62
10 a)	2	0	1.00	18	0.90	43	1.01	69	1.01	130	0.97	166	0.99	456	0.98	746	0.98
10	4	0	1.00	13	0.83	36	0.74	63	0.77	124	0.66	161	0.69	448	0.63	737	0.55
10	4	0	1.00	15	0.97	39	0.97	66	0.91	127	0.86	164	0.84	452	0.71	742	0.63
10 a)	4	0	1.00	19	0.87	43	0.87	69	0.99	130	0.99	167	0.99	457	0.99	746	0.98

a) Set of data not used in the statistical analyses

APPENDIX F

POLYNOMIAL REGRESSION EQUATIONS

Table F-1. Polynomial Regression Equations

The equations will be in the form:

$$r = C_0 + C_1T + C_2T^2 + C_3T^3 + C_4T^4 + C_5T^5$$

where r_h is the hydraulic head ratio

T is the time in minutes

C_0 is the intercept

C_0, C_1, C_2, C_3, C_4, C_5 are the statistically significant regression coefficients

				Polynomial Regression Coefficient				
Location	Depth (ft)	Distance (ft)	C_0	C_1	C_2	C_3	C_4	C_5
SW	6	2	0.962	-0.125×10^{-1}	0.477×10^{-4}	-0.399×10^{-7}	-0.771×10^{-10}	0.973×10^{-13}
NW	6	2	0.875	-0.315×10^{-2}	0.274×10^{-5}			
NE	6	2	1.021	-0.874×10^{-2}	0.205×10^{-4}	-0.141×10^{-7}		
SE	6	2	0.946	-0.890×10^{-2}	0.226×10^{-4}	-0.166×10^{-7}		
SW	6	4	0.788	-0.358×10^{-2}	-0.410×10^{-4}	0.389×10^{-6}	-0.957×10^{-9}	0.696×10^{-12}

Table F-1. (continued)

Location	Depth (ft)	Distance (ft)	C_0	C_1	C_2	C_3	C_4	C_5
NW	6	4	1.003	-0.740×10^{-2}	0.185×10^{-4}	-0.139×10^{-7}		
NE	6	4	0.935	0.111×10^{-2}	-0.617×10^{-4}	0.179×10^{-6}	-0.134×10^{-9}	
SE	6	4	1.001	-0.734×10^{-2}	0.164×10^{-4}	-0.112×10^{-7}		
SW	7	2	0.743	-0.619×10^{-2}	0.204×10^{-4}	-0.184×10^{-7}		
NW	7	2	0.943	-0.786×10^{-2}	0.469×10^{-4}	-0.101×10^{-6}	0.679×10^{-10}	
NE	7	2	0.768	-0.398×10^{-3}				
SE	7	2	0.886	-0.335×10^{-2}	0.869×10^{-5}	-0.665×10^{-8}		
SW	7	4	0.792	-0.408×10^{-2}	0.113×10^{-4}	-0.896×10^{-8}		
NW	7	4	0.965	-0.499×10^{-2}	0.213×10^{-4}	-0.136×10^{-7}	-0.602×10^{-10}	0.675×10^{-13}
NE	7	4	0.894	-0.139×10^{-2}	0.112×10^{-5}			
SE	7	4	0.905	-0.310×10^{-2}	0.781×10^{-5}	-0.587×10^{-8}		
SW	8	2	0.972	-0.154×10^{-2}	0.111×10^{-4}	-0.281×10^{-7}	0.208×10^{-10}	

Table F-1. (continued)

Location	Depth (ft)	Distance (ft)	C_0	C_1	C_2	C_3	C_4	C_5
NW	8	2						
NE	8	2	0.932	-0.394×10^{-3}				
SE	8	2	0.798	-0.422×10^{-3}				
SW	8	4	0.790	-0.403×10^{-2}	0.113×10^{-4}	-0.989×10^{-8}		
NW	8	4	0.982	-0.593×10^{-2}	0.352×10^{-4}	-0.762×10^{-7}	0.517×10^{-10}	
NE	8	4	0.845	-0.385×10^{-3}				
SE	8	4	0.910	-0.259×10^{-2}	0.608×10^{-5}	-0.441×10^{-8}		
SW	9	2	0.788	-0.498×10^{-3}				
NW	9	2	0.895	-0.376×10^{-3}				
NE	9	2						
SE	9	2						
SW	9	4	0.879	-0.335×10^{-3}				

Table F-1. (continued)

Location	Depth (ft)	Distance (ft)	C_0	C_1	C_2	C_3	C_4	C_5
NW	9	4	0.968	-0.248×10^{-2}	0.514×10^{-5}	-0.346×10^{-8}		
NE	9	4	0.909	-0.131×10^{-2}	0.108×10^{-5}			
SE	9	4						
SW	10	2						
NW	10	2	0.877	-0.166×10^{-2}	0.154×10^{-5}			
NE	10	2	0.971	-0.471×10^{-3}				
SE	10	2	0.912	-0.113×10^{-2}	0.941×10^{-6}			
SW	10	4	0.881	-0.319×10^{-2}	0.808×10^{-5}	-0.616×10^{-8}		
NW	10	4	0.902	-0.335×10^{-3}				
NE	10	4	0.918	-0.375×10^{-3}				
SE	10	4	0.886	-0.445×10^{-3}				

APPENDIX G

Plots of Hydraulic Head Ratios Versus Time

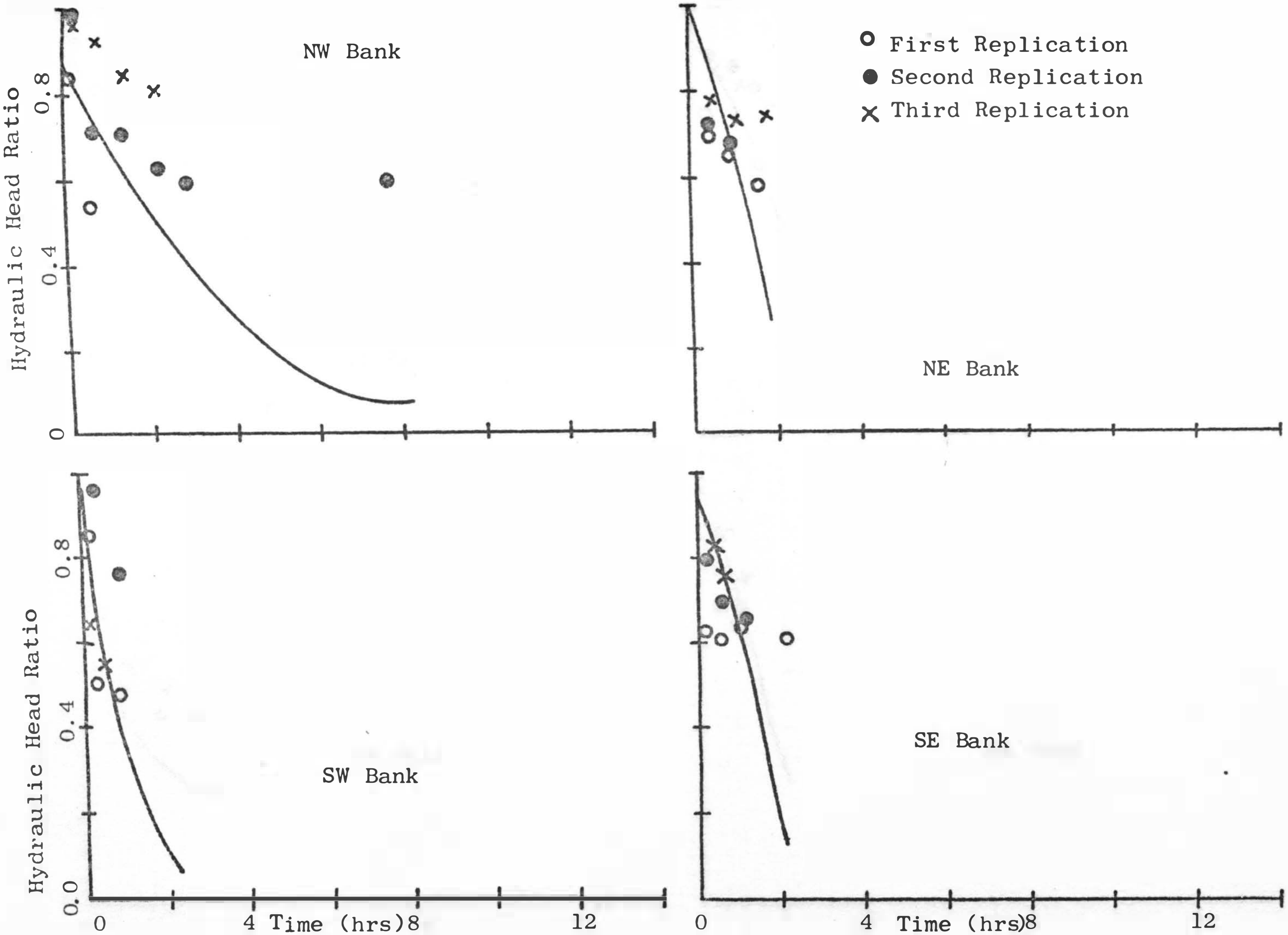

Figure 14. Plots of Hydraulic Head Ratios Versus Time for the Six Foot Piezometers at the Two Foot Distance

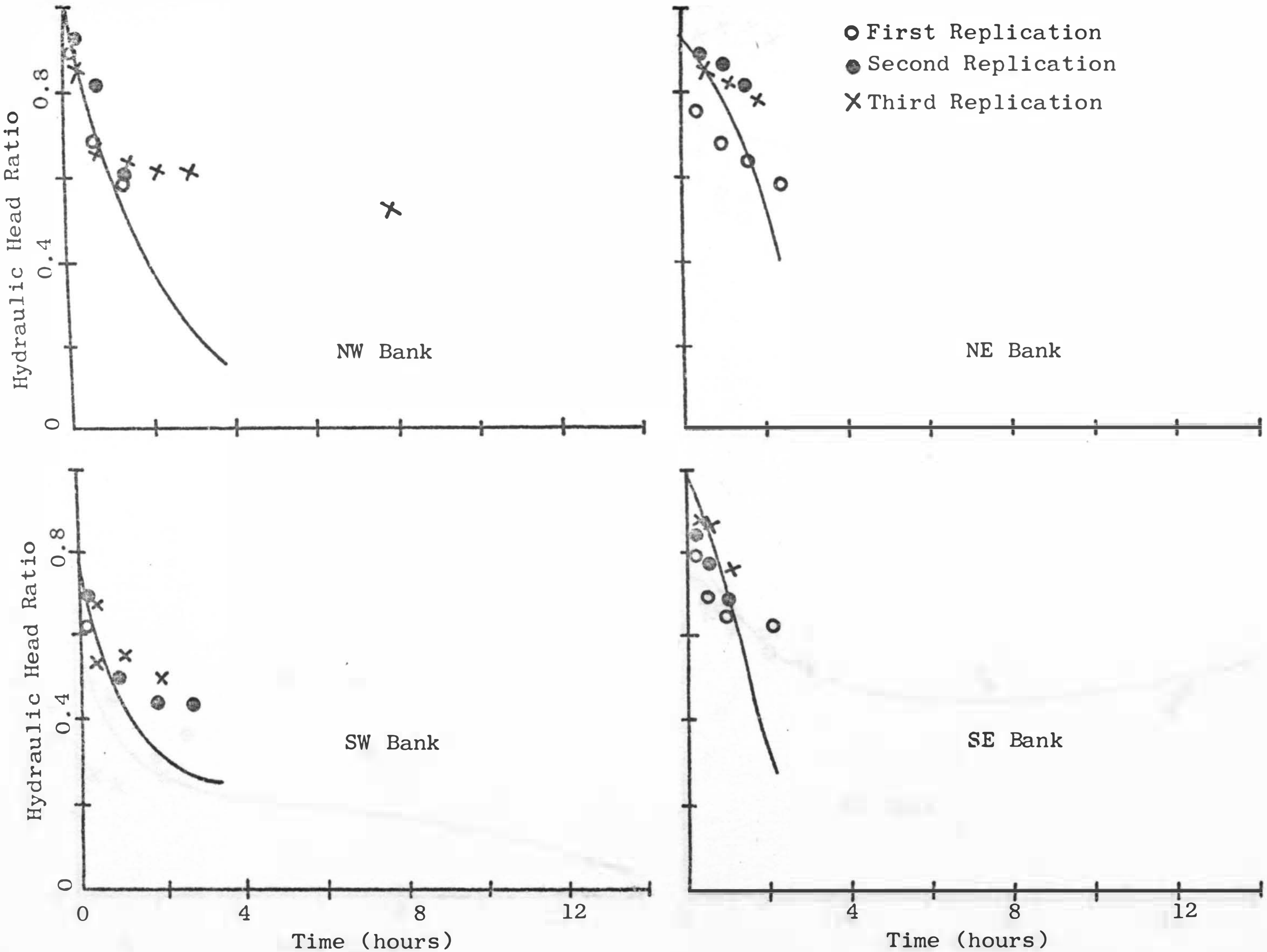

Figure 15. Plots of Hydraulic Head Ratios Versus Time for the Six Foot Piezometers at the Four Foot Distance

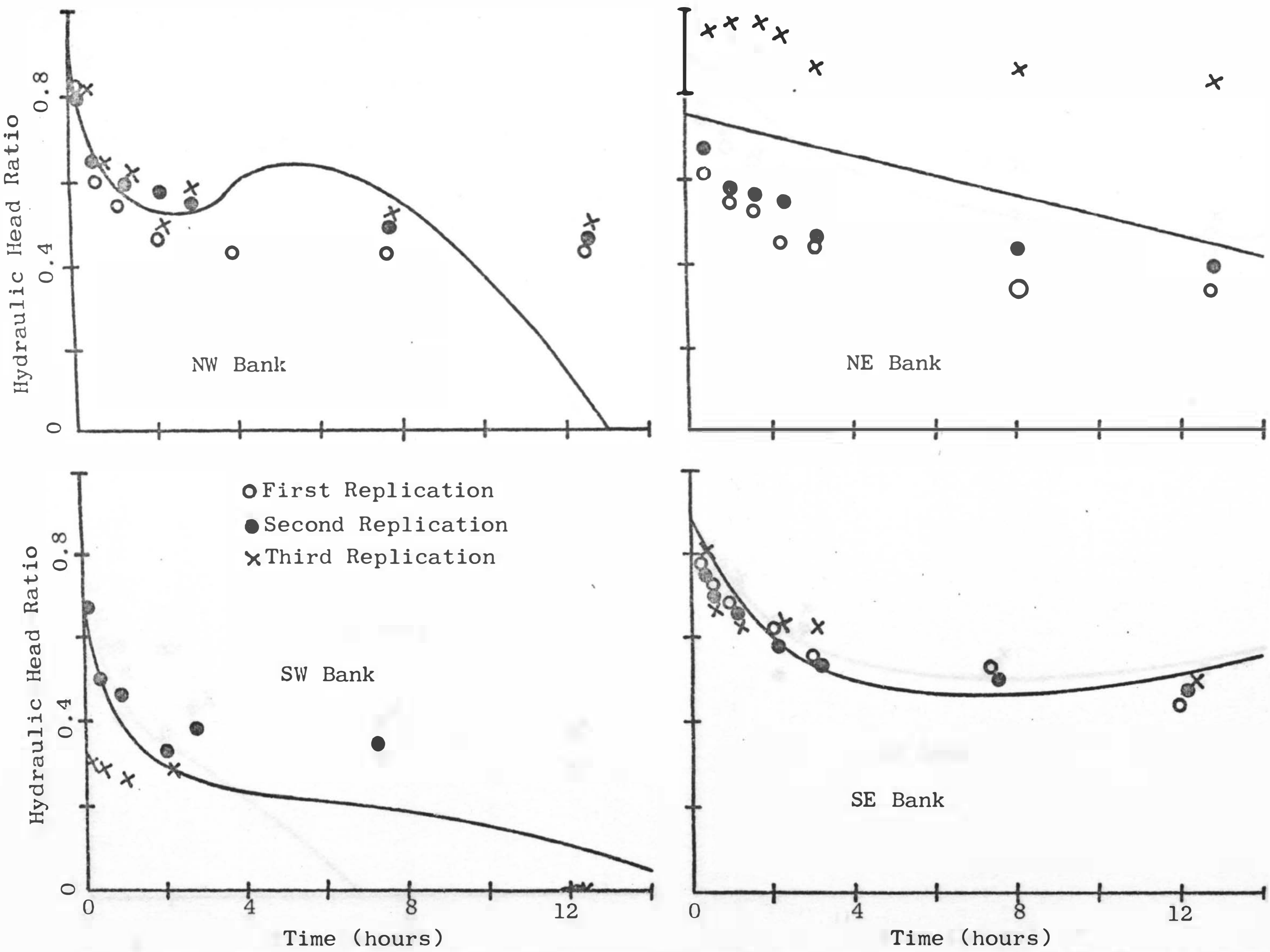

Figure 16. Plots of Hydraulic Head Ratios Versus Time for the Seven Foot Piezometers at the Two Foot Distance

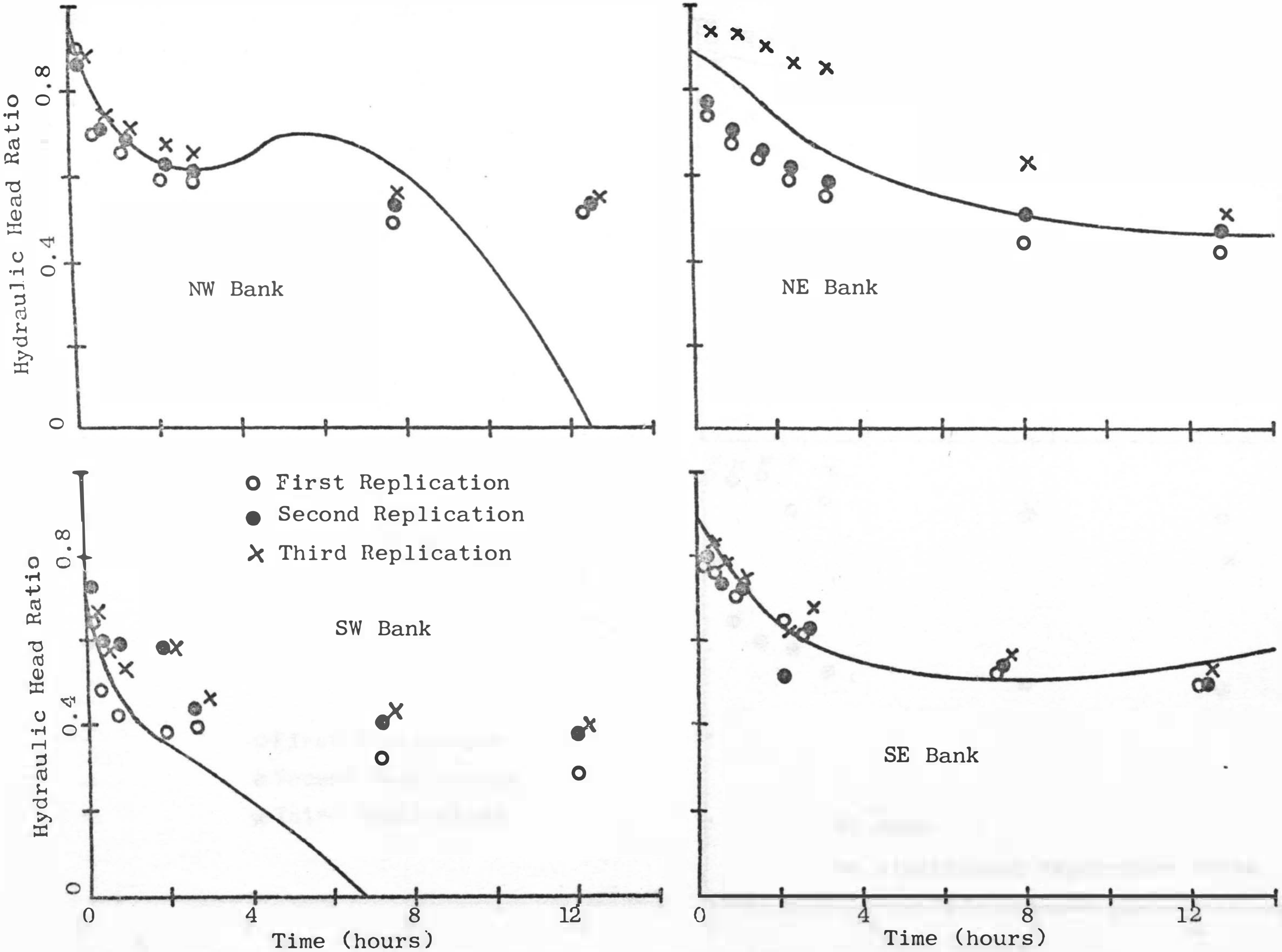

Figure 17. Plots of Hydraulic Head Ratios Versus Time for the Seven Foot Piezometers at the Four Foot Distance

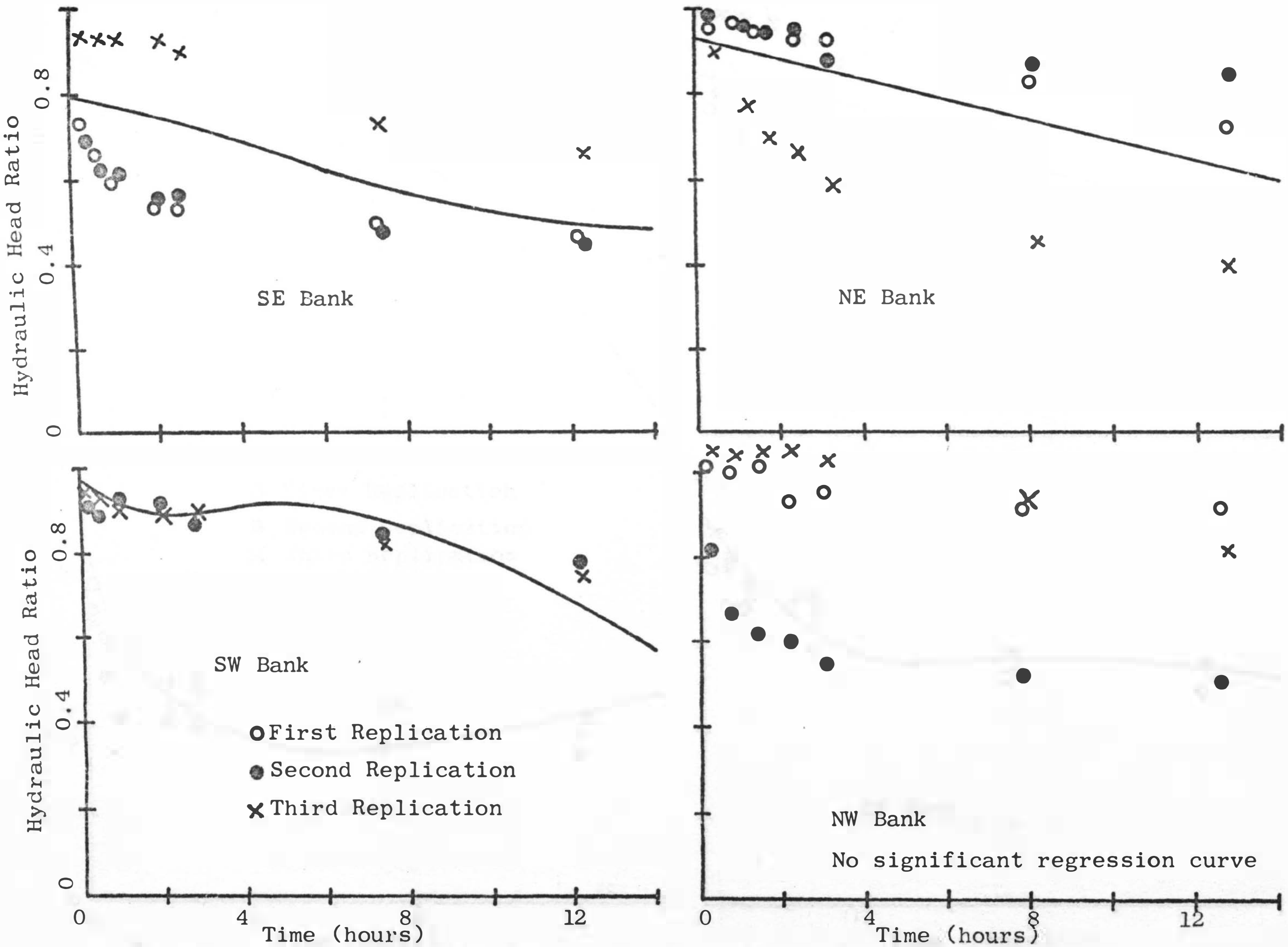

Figure 18. Plots of Hydraulic Head Ratios Versus Time for the Eight Foot Piezometers at the Two Foot Distance

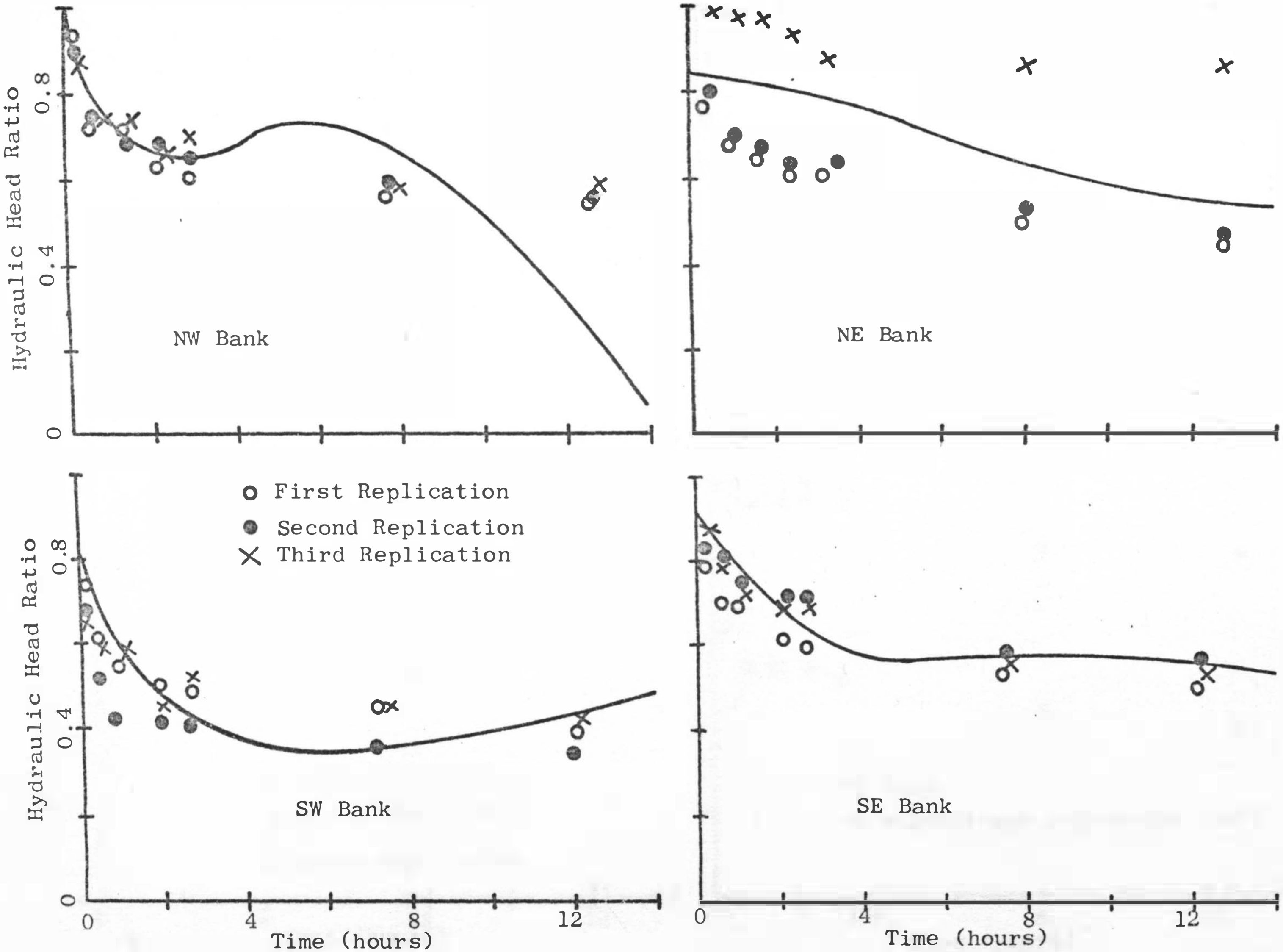

Figure 19. Plots of Hydraulic Head Ratios Versus Time for the Eight Foot Piezometers at the Four Foot Distance

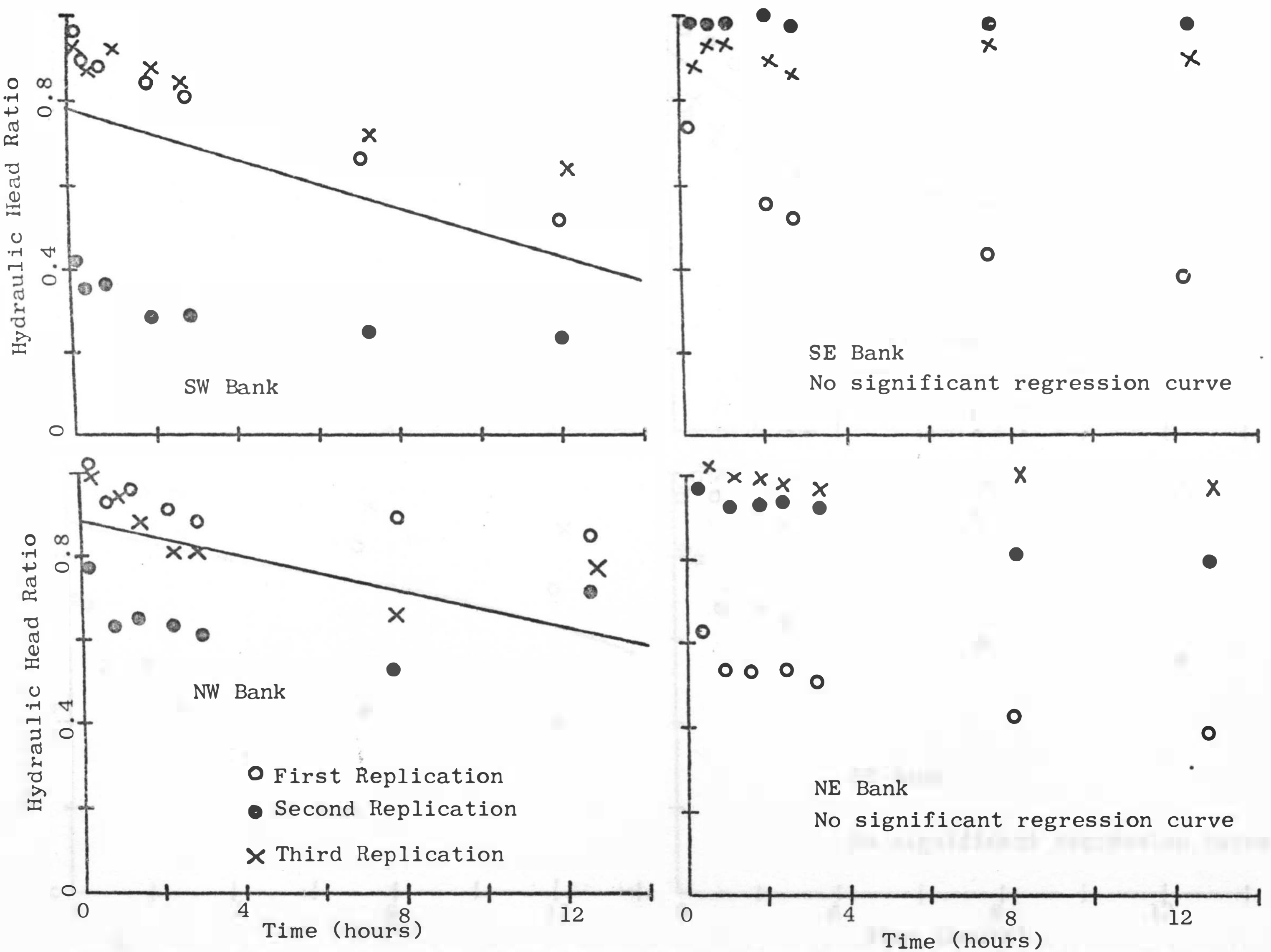

Figure 20. Plots of Hydraulic Head Ratios Versus Time for the Nine Foot Piezometers at the Two Foot Distance

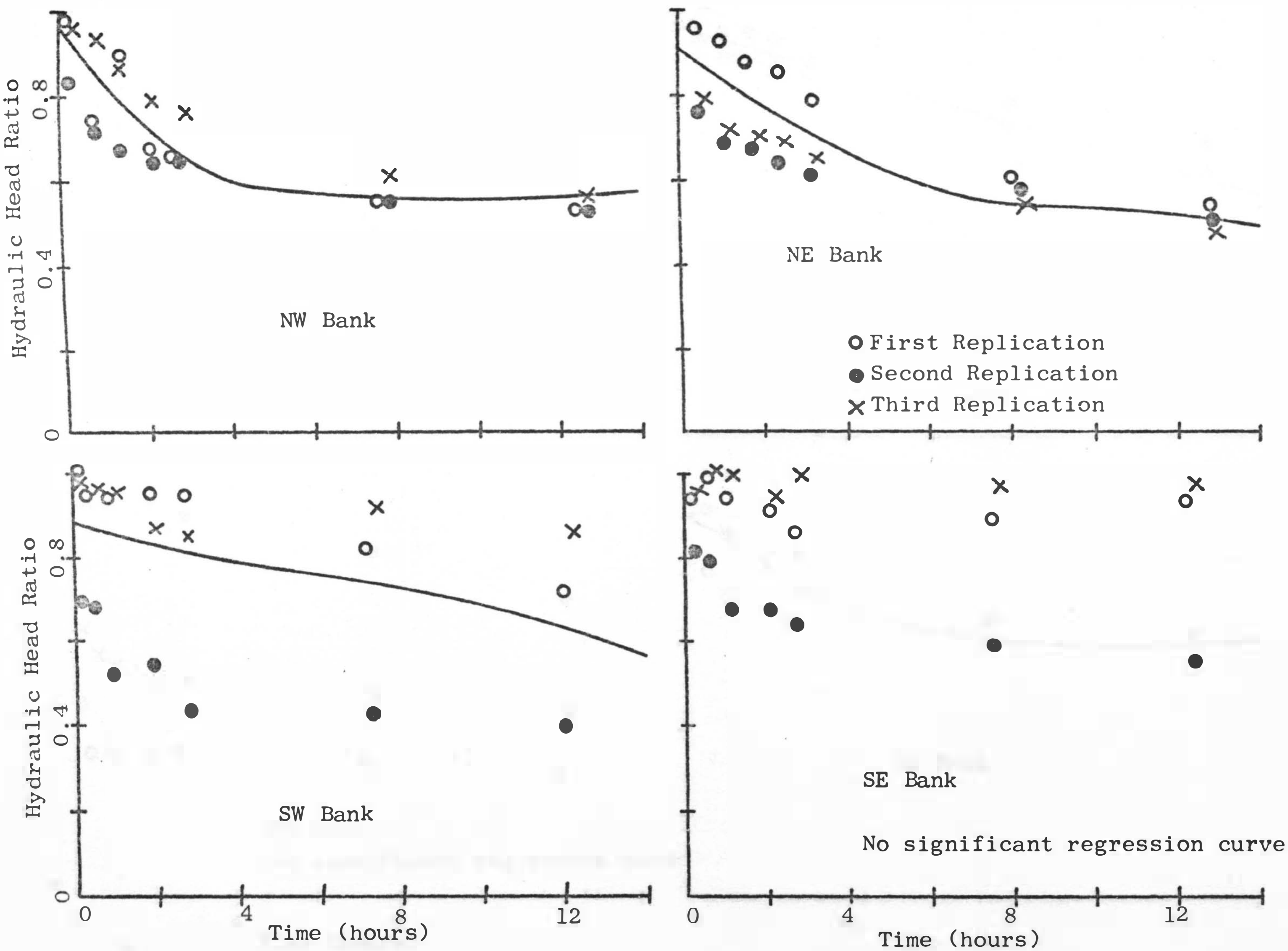

Figure 21. Plots of Hydraulic Head Ratios Versus Time for the Nine Foot Piezometers at the Four Foot Distance

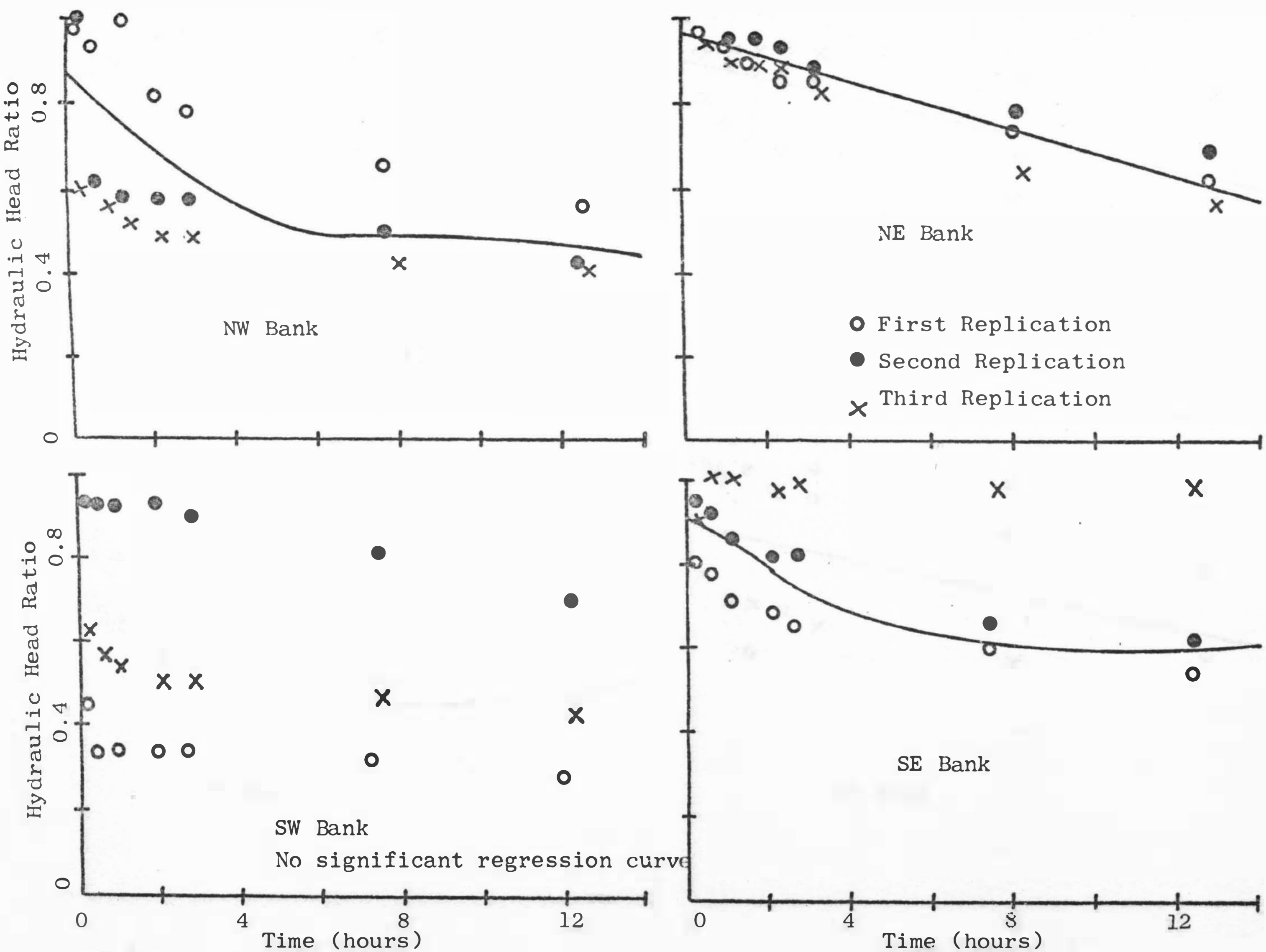

Figure 22. Plots of Hydraulic Head Ratios Versus Time for the Ten Foot Piezometers at the Two Foot Distance

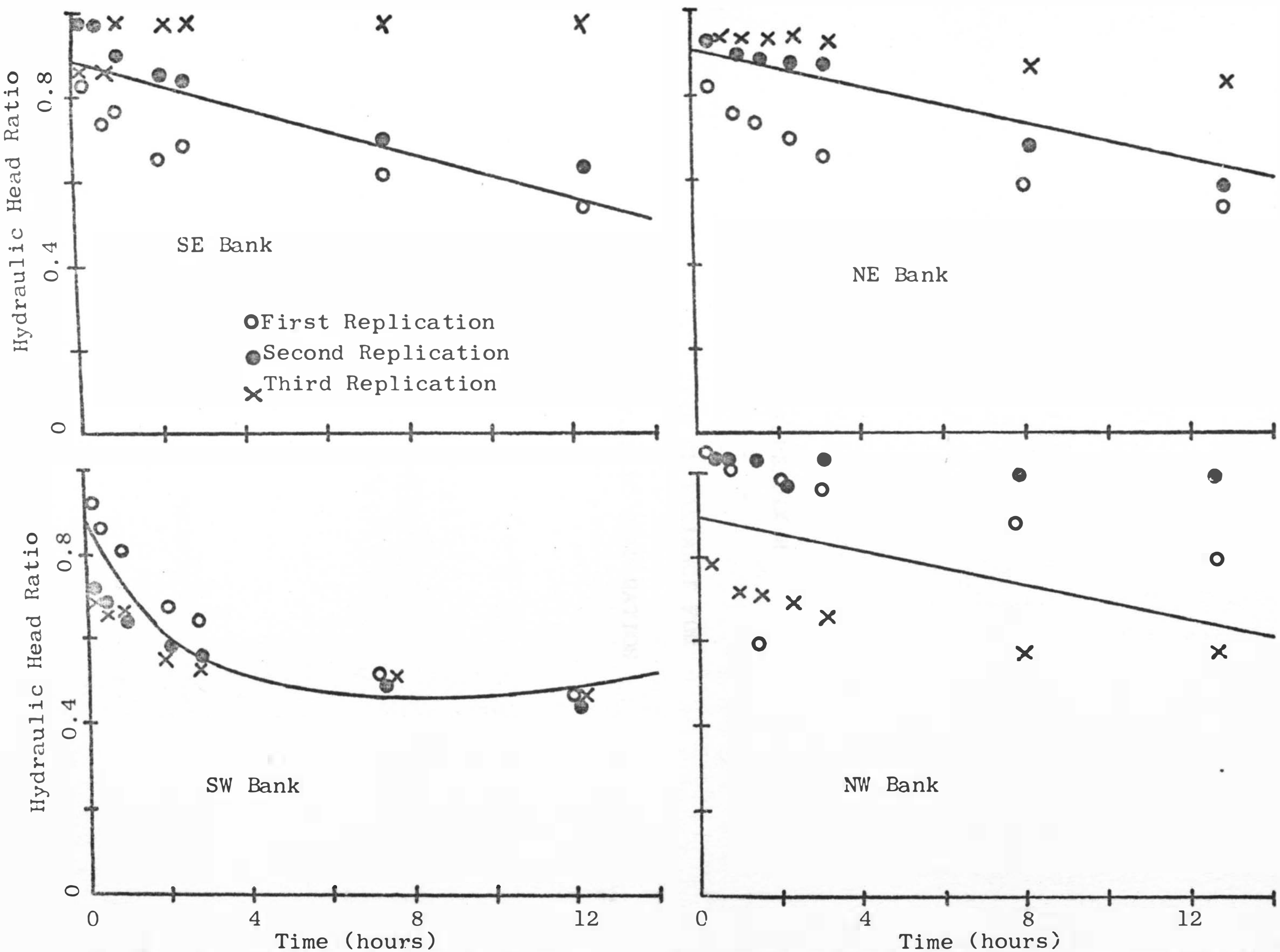

Figure 23. Plots of Hydraulic Head Ratios Versus Time for the Ten Foot Piezometers at the Four Foot Distance

APPENDIX H

ANALYSES OF VARIANCE FOR THE HYDRAULIC HEAD RATIOS

Table H-1. Analysis of Variance for the Piezometric Data of Time Block 1

Source	D.F.	Sum of Squares	Mean Squares	F
Soil	1	0.094	0.094	5.70*
Envelope	1	0.109	0.109	6.60*
Depth	3	0.483	0.161	9.74***
Replication	2	0.045	0.022	1.35
Distance	1	0.007	0.007	0.42
Soil x Envelope	1	0.216	0.216	13.06***
Soil x Depth	3	0.122	0.041	2.46
Soil x Distance	1	0.033	0.033	1.98
Envelope x Depth	3	0.126	0.042	2.54
Envelope x Distance	1	0.029	0.029	1.75
Depth x Distance	3	0.237	0.079	4.77***
Soil x Envelope x Depth	3	0.119	0.040	2.39
Soil x Envelope x Distance	1	0.00+	0.00+	0.01
Soil x Depth x Distance	3	0.111	0.037	2.25
Envelope x Depth x Distance	3	0.167	0.056	3.37*
Soil x Envelope x Depth x Distance	3	0.059	0.019	1.14
Error	100	1.652	0.017	

*** Significant at the .005 level

* Significant at the .05 level

Table H-2. Analysis of Variance for the Piezometric Data of Time Block 2

Source	D.F.	Sum of Squares	Mean Squares	F
Soil	1	0.148	0.148	7.70***
Envelope	1	0.056	0.056	2.93
Depth	3	0.611	0.204	31.87***
Replication	2	0.008	0.004	0.19
Distance	1	0.004	0.004	0.20
Soil x Envelope	1	0.272	0.272	14.20***
Soil x Depth	3	0.171	0.057	3.00*
Soil x Distance	1	0.003	0.003	0.17
Envelope x Depth	3	0.072	0.024	1.30
Envelope x Distance	1	0.00+	0.00+	0.01
Depth x Distance	3	0.342	0.114	5.90***
Soil x Envelope x Depth	3	0.128	0.042	2.20
Soil x Envelope x Distance	1	0.005	0.005	0.26
Soil x Depth x Distance	3	0.168	0.056	2.90*
Envelope x Depth x Distance	3	0.116	0.039	2.01
Error	96			

*** Significant at the .005 level

*   Significant at the .05 level

Table H-3. Analysis of Variance for the Piezometric Data of Time Block 3

Source	D.F.	Sum of Squares	Mean Squares	F
Soil	1	0.264	0.264	11.64***
Envelope	1	0.055	0.055	2.42
Depth	3	1.187	0.396	17.44***
Replication	2	0.071	0.035	1.56
Distance	1	0.016	0.016	0.71
Soil x Envelope	1	0.213	0.213	9.37***
Soil x Depth	3	0.219	0.073	3.22*
Soil x Distance	1	0.017	0.017	0.76
Envelope x Depth	3	0.117	0.039	1.73
Envelope x Distance	1	0.003	0.003	0.16
Depth x Distance	3	0.539	0.180	7.92***
Soil x Envelope x Depth	3	0.202	0.067	2.96
Soil x Envelope x Distance	1	0.010	0.010	0.43
Soil x Depth x Distance	3	0.303	0.101	4.46***
Envelope x Depth x Distance	3	0.252	0.084	3.70*
Error	144	3.265	0.022	

*** Significant at the .005 level

*   Significant at the .05 level

Table H-4. Analysis of Variance for the Piezometric Data of Time Block 4

Source	D.F.	Sum of Squares	Mean Squares	F
Soil	1	0.046	0.046	1.88
Envelope	1	0.030	0.030	1.23
Depth	3	0.283	0.094	3.86*
Replications	2	0.027	0.013	0.55
Distance	1	0.030	0.030	1.22
Soil x Envelope	1	0.035	0.035	1.43
Soil x Depth	3	0.112	0.037	1.53
Soil x Distance	1	0.00+	0.00+	0.0+
Envelope x Depth	3	0.074	0.024	1.00
Envelope x Distance	1	0.003	0.003	0.11
Depth x Distance	3	0.156	0.052	2.12
Soil x Envelope x Depth	3	0.060	0.020	0.81
Soil x Envelope x Distance	1	0.011	0.011	0.44
Soil x Depth x Distance	3	0.114	0.038	1.56
Envelope x Depth x Distance	3	0.093	0.031	1.27
Error	55	1.345	0.024	

* Significant at the .05 level

Table H-5. Analysis of Variance for the Piezometric Data of Time Block 5

Source	D.F.	Sum of Squares	Mean Squares	F
Soil	1	0.055	0.055	2.46
Envelope	1	0.023	0.023	1.02
Depth	3	0.240	0.080	3.61*
Replication	2	0.027	0.014	0.62
Distance	1	0.021	0.021	0.95
Soil x Envelope	1	0.012	0.012	0.56
Soil x Depth	3	0.066	0.022	0.99
Soil x Distance	1	0.010	0.010	0.45
Envelope x Depth	3	0.063	0.021	0.94
Envelope x Distance	1	0.021	0.021	0.94
Depth x Distance	3	0.211	0.070	3.18*
Soil x Envelope x Depth	3	0.061	0.020	0.91
Soil x Envelope x Distance	1	0.001	0.001	0.04
Soil x Depth x Distance	3	0.135	0.045	2.03
Envelope x Depth x Distance	3	0.064	0.021	0.96
Error	55	1.218	0.022	

* Significant at the .05 level

APPENDIX I

MECHANICAL ANALYSIS DATA FOR THE FIELD GRAVEL ENVELOPE SAMPLES

Table I-1. Particle Size Distribution (Percent Finer by Weight) for the Field Gravel Envelope Samples

Envelope Position	Sample Number	Sieve Size (mm)							
		4.760	1.190	0.841	0.420	0.250	0.177	0.275	0.053
Top	1	79.3	29.5	17.9	5.4	2.2	1.4	0.6	0.3
Top	2	79.9	27.3	16.8	5.4	2.4	1.6	0.8	0.4
Top	3	80.6	24.9	15.7	5.3	2.1	1.4	0.5	0.5
Top	4	84.9	30.2	19.2	5.9	2.1	1.3	0.7	0.3
Top	5	66.3	27.9	18.7	7.0	3.0	1.9	0.6	0.4
Top	6	61.1	22.1	14.2	5.1	2.2	1.4	0.7	0.4
Top	7	70.7	26.1	17.2	7.0	3.6	2.5	1.0	0.2
Top	8	83.7	28.8	17.7	5.9	2.8	2.0	1.1	0.5
Top	9	73.5	25.4	16.5	6.7	3.7	2.8	1.7	1.1
Top	10	73.0	26.7	14.1	5.3	2.9	2.2	1.3	0.5
Top	11	75.4	23.2	14.5	5.2	2.7	1.9	0.9	0.4
Top	12	58.5	25.8	17.6	7.1	3.5	2.4	1.1	0.5
Side	13	87.4	30.7	19.6	6.3	2.5	1.6	0.7	0.4
Side	14	80.9	29.5	17.5	5.4	2.0	1.3	0.5	0.3
Side	15	77.0	25.9	16.2	5.1	1.8	1.1	0.4	0.3
Side	16	76.0	28.8	19.3	9.1	5.7	4.6	3.1	0.9
Side	17	68.6	26.6	17.8	7.1	3.4	2.2	1.0	0.7
Bottom	18	63.0	20.3	13.3	5.2	2.4	1.7	0.7	0.3
Bottom	19	49.8	21.2	14.9	6.7	3.5	2.4	1.2	0.9
Bottom	20	67.6	30.3	23.1	14.3	10.2	8.3	5.6	1.3
Averages									
Top		73.9	26.2	16.7	5.9	2.8	1.9	0.9	0.5
Side		77.9	28.3	18.1	6.6	3.1	2.2	1.1	0.5
Bottom		60.1	23.9	17.1	8.7	5.4	4.1	2.5	0.8
Total		72.8	26.4	17.1	6.5	3.2	2.3	1.2	0.5

APPENDIX J

LABORATORY ANALYSIS DATA

FOR THE FIBERGLASS SAMPLES

Table J-1. Hydraulic Conductivity (cm/sec) for the Fiberglass Envelope Material

Control (Unused) Sample		Field Samples
0.315		0.199
0.492		0.236
0.367		0.211
0.350		0.399
0.632		0.321
		0.195
		0.217
		0.309
		0.314
		0.457
		0.109
		0.134
		0.263
		0.370
		0.337
		0.326
		0.312
		0.209
		0.160
		0.209
		0.247
		0.346
		0.236
		0.336
		0.270
		0.514
		0.253
0.431	Average	0.277

Table J-2. Tensile Strength (grams/centimeter of width) for the Fiberglass Envelope Material

Control (Unused) Sample		Field Samples
612.8		274.2
757.5		688.6
823.3		396.6
805.0		327.4
917.5		479.2
		548.5
		158.5
		862.0
		570.3
		712.5
		919.8
		396.4
		601.4
		344.0
		780.5
		100.0
		391.5
		357.5
		189.1
		680.1
		247.5
		687.1
		311.8
		474.3
783.2	Average	479.1